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Fifth International Conference on Low-Volume Roads 1991

Papers presented at the Fifth International Conference on Low-Volume Roads May 19–23, 1991 Raleigh, North Carolina

A peer-reviewed publication of the Transportation Research Board

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. 1991

Transportation Research Record 1291
Price: $84.00

Subscriber Categories
I planning, administration, and environment
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Printed in the United States of America

Library of Congress Cataloging-in-Publication Data
National Research Council. Transportation Research Board.

International Conference on Low-volume Roads (5th : 1991 : Raleigh, N.C.)
Fifth International Conference on Low-volume Roads : papers presented at the Fifth International Conference on Low-Volume Roads, May 19–23, 1991, Raleigh, North Carolina.
p. cm.—(Transportation research record, ISSN 0361-1981 ; no. 1291)
ISBN 0-309-05071-5
1. Low-volume roads. I. National Research Council (U.S.). Transportation Research Board. II. Title. III. Title: 5th International Conference on Low-volume Roads. IV. Series: Transportation research record ; 1291.
TE7.H5 no. 1291
[TE228.5]
388 s—dc20
[625.7]

91-8965
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Foreword

Too often low-volume roads are the last part of the road system to benefit from the latest engineering ideas. They seldom have been the object of research. Like poor cousins, they are too numerous and their problems too exacting for the limited resources of most of the world's major highway research agencies. Public works departments are conservative, and local roads agencies seem to be the most conservative and most poorly financed of all, so innovations often must wait until they are fully proven on major highways before they are used on low-volume roads.

To focus attention on this problem, in 1975 the first International Low-Volume Roads Conference was convened in Boise, Idaho, by the Transportation Research Board. The late Eldon Yoder of Purdue University provided the leadership to get the series of conferences under way. Since then, conferences have been held every 4 years. Their sites have included Ames, Iowa; Phoenix, Arizona; Ithaca, New York; and now Raleigh, North Carolina.

The conferences provide a forum that will move low-volume roads out of the poor cousin category. They have brought together major researchers and innovative practitioners to talk and to listen to each other. Whereas the conferences provide a meeting place for a few hundred people for a few days, these proceedings provide detailed communication to thousands of interested engineers and planners for many years following each conference.

These proceedings represent the combined effort of hundreds of people. Surely the greatest recognition should go to the authors of the papers printed in these two volumes. The credit for doing the research and documenting it is theirs alone, and we are deeply indebted to them. In addition to reviewing every detail of the conference plans, the conference steering committee spent several days reviewing synopses and choosing the papers that appear in these proceedings.

The papers underwent peer review. Nearly 500 reviewers offered suggestions on how the authors could improve their communications. A staff of editors at TRB spent many hours clarifying the language and putting the papers into standard form. Keeping track of all of the papers as they moved from initial synopsis to final manuscript was no small task, and credit for this goes to G. P. Jayaprakash and his secretaries at TRB.

None of this would have been possible without the financial support of the conference sponsors: the U.S. Department of Agriculture Forest Service, the Federal Highway Administration, the Bureau of Indian Affairs, and the U.S. Army Corps of Engineers. Though the conference participants paid the direct costs of the meeting in North Carolina, the sponsors funded the costs incurred by TRB for editing and printing the proceedings and publicizing, general management, and organization of the conference. Also, the conference hosts, the Institute for Transportation Research and Education of the University of North Carolina, organized the field trip and provided excellent facilities and staff support for the meeting.

To all of these many people I am deeply indebted. I extend my heartfelt thanks to each one.

Lynne H. Irwin
Steering Committee Chairman

Economics and Finance

Financing Low-Volume Roads and Bridges: Results from a National Survey

NORMAN WALZER AND RUTH T. MCWILLIAMS

Financing low-volume roads, usually the responsibility of counties, towns, or townships, recently has been a subject of interest and some concern for state and local policymakers. Low-volume roads and bridges represent the main connection between residents in unincorporated areas and essential public services, such as ambulances and fire trucks. In recent years, however, the process of financing the off-system transportation network has changed, with local resources becoming a greater share of the overall financing system. The condition of the rural transportation network is examined on the basis of a mail survey of county highway officials. Financing arrangements also are reviewed, with special attention paid to the growing reliance on local resources in the financing arrangements. Finally, options available to local highway officials for meeting their perceived needs are examined. According to survey respondents, the condition of low-volume roads and bridges needs attention and repair of deficiencies may require additional funding. Many respondents report that budgets are inadequate to provide the services that they consider necessary. Although recent population trends have rendered some of the local mileage no longer economically feasible, incentive remains small for either residents or local officials to remove road mileage from the inventory.

Low-volume roads and bridges in rural areas have been a policy issue for rural residents and policy makers for many years. Rural farm residents need access to markets for their products and, increasingly, to off-farm employment. Rural nonfarm residents require regular access to employment in nearby communities and many rural residents depend on the local transportation network for public services, such as fire protection and emergency medical care.

However, rural states face several major issues in financing roads and bridges. First, changes in agricultural methods involving heavier farm equipment have increased demands on the local transportation network. Semitrailers weighing more than 60,000 lb travel off-system roads and bridges. The changes have occurred concurrently with population declines, forcing infrastructure costs to be paid by a smaller number of residents, and resulting in increased cost per resident.

Second, the relatively poor performance of the farm economy in the early 1980s, with subsequent income declines, meant that property taxes traditionally used to finance roads and bridges increased relative to farmers' ability to pay. In some instances, pressures by the agricultural community for property tax relief affected the ability of local governments to finance public services.

Third, federal general revenue sharing was eliminated beginning in 1987, removing a revenue source commonly used to finance roads and bridges. Other local revenues, such as sales taxes, did not perform well in many rural areas during the early and mid-1980s and, as a result, local officials have had to find alternative revenues such as user charges or local imposts.

Engineering condition data for off-system roads and bridges are not available on a national basis. However, information on the estimated condition of roads and bridges is available from a 1987 national survey of county highway superintendents conducted by the National Association of Counties (*1,2*). Although these estimates lack the precision of engineering analysis, many states require that county highway superintendents be engineers. They are therefore in the most informed position to evaluate the ability of roads and bridges to meet local transportation demands and determine spending patterns and revenue sources of local governments.

Recent shifts in revenue sources have placed greater burden on local revenue sources. The additional burden in rural areas facing economic declines makes the financing of roads and bridges at current levels more difficult although, as will be shown later, real spending has increased in the past decade.

Correlation analysis is used to examine relationships between conditions of roads and bridges, and financing patterns. The relationships between spending patterns, road density, budget outlook, and condition are considered.

Although specific policy recommendations are not made, local policy makers should evaluate policy alternatives. Each option has difficulties associated with its implementation, but given the trends, a long-term financing solution may require some of these alternatives. Changes in state distribution programs may be required to allow local governments to respond better to changing population and economic conditions.

Local roads and bridges include those off the federal aid highway system and because most of the mileage is in nonmetropolitan areas, the terms "rural" and "low-volume" refer to the same road mileage. In 1987, there were 3,020,000 mi of off-system roads and 302,011 bridges longer than (or equal to) 20 ft (*3*).

Subsequent analyses use information from a 1987 national mail survey of 533 counties conducted in cooperation with the National Association of Counties (NAC) (*1,2*). The response rate for counties was 25.4 percent. A similar survey was conducted by the National Association of Towns and Townships,

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with a response rate of 14.8 percent of the towns or townships that have direct responsibility for roads and bridges, and these results are reported elsewhere (*1,2*). Financial information is obtained from FHWA's *Highway Statistics* (*3*) for counties and townships combined and from the 1987 Census of Governments finance data file for counties.

Every 2 years (within 6 months), state and local public officials inspect bridges of 20 ft and longer and report the ratings to the FHWA. Although the National Bridge Inventory (NBI) has been criticized, it is still a major information source on bridge conditions. No such comprehensive data source exists for roads.

LOCAL ROAD AND BRIDGE CONDITIONS

The views of local highway officials concerning road conditions are especially useful because these decision makers are responsible for financing local roads and bridges. If, in their perception, additional funding is needed, they can take appropriate action; if, on the other hand, the mileage can be reduced, they can adopt that strategy as well.

County highway officials rated the mileage under their responsibility using a 10-point scale, ranging from 0, for a road that is closed, to 9, for a road that is in new or perfect condition. This rating system is in common use and is familiar to many county highway officials. Each rating category contains a brief description of road condition to maximize reporting comparability among responses. The rating system is presented in Table 1.

On average, 1 mi in 5 on the county highway system is perceived as less than barely adequate, a rating of 4 or below (Table 1). An additional 12.7 percent of the county highway system was rated as having limited failures and barely adequate, a rating of 5, bringing the total to 35.6 percent as barely adequate or worse in the view of county highway officials. Information is not available regarding whether the mileage is considered inadequate for volumes of traffic, weight, or both.

Regions differ in reported road conditions. Counties in Pacific states reported 40.1 percent as barely adequate or below (Table 1), whereas those in the Northeast reported an average of 28.9 percent. Available information fails to indicate reasons for these differences and further work is needed to determine whether they are significant between regions.

County bridges also have problems, on the basis of the NBI (*4*). Bridges are rated on 100 points, using a system of deficiency points. If a bridge receives a Sufficiency Rating (SR) of >80, it does not qualify for federal funding under the Highway Bridge Replacement and Rehabilitation Program (HBRRP). If rated between SR 50 and 80, a bridge qualifies for rehabilitation, but not replacement. Bridges that are to remain in use with ratings of <50 qualify for replacement or rehabilitation. The NBI, however, has been criticized for including bridges not intended for future use, thereby implying a state's bridge condition is worse than it actually is (*5*).

Nationwide, 22.5 percent of county-maintained bridges had SR values of >80 (Table 2). These bridges are judged adequate for expected traffic and loads. Although they do not qualify for funding through the HBRRP, they may receive state funds. Judging bridge adequacy is complicated by the

TABLE 1 CONDITIONS OF COUNTY ROADS BY REGION

		Northeast (n=43)		Midwest (n=157)		Appalachian (n=37)		Southern (n=96)		Mountain States (n=56)		Pacific States (n=79)		U.S. Counties (n=468)	
Condition Description	Rating	Pct.	Avg. Miles	Pct.	Avg. Miles	Pct.	Avg. Miles	Pct.	Avg. Miles	Pct.	Avg. Miles	Pct.	Avg. Miles	Pct.	Avg. Miles
New or Perfect Condition	9	9.5	31.2	4.4	47.2	9.9	46.9	8.9	65.8	5.0	40.6	6.4	65.3	6.6	51.6
	8	14.4	100.8	5.9	53.1	6.1	12.9	7.3	65.8	3.9	43.2	7.3	61.6	7.0	55.6
Surface adequate with normal maintenance	7	33.8	156.0	39.9	418.2	42.8	212.4	42.1	320.3	45.2	401.2	37.7	350.8	40.3	338.2
	6	13.4	85.2	13.4	120.6	11.7	75.8	6.3	63.8	9.2	118.2	8.5	83.7	10.5	94.6
Limited failures and barely adequate	5	12.9	47.0	11.3	93.1	8.9	41.2	16.5	133.9	12.0	107.5	12.9	131.2	12.7	98.7
Maintenance will be considerably higher to prevent continued deterioration	4	10.0	35.3	12.4	110.4	12.0	56.3	11.5	96.3	13.3	127.5	16.4	180.4	12.7	107.1
Considerable failures and beyond practical limits of normal maintenance	3	2.3	7.0	5.0	42.9	2.5	27.7	2.8	23.5	3.5	25.7	4.4	49.2	3.8	32.9
	2	3.4	11.4	5.5	61.0	5.0	24.8	3.3	41.2	5.8	47.7	4.6	43.0	4.7	44.0
Failure to the extent that operation of traffic is severely affected	1	.3	.8	1.9	16.4	.9	6.8	1.2	8.7	1.9	17.1	1.7	21.2	1.5	13.1
Closed, awaiting repairs	0	-	-	.3	2.8	.2	.8	.1	.7	.2	2.9	.1	1.4	.2	1.7
Totals		100.0	474.8	100.0	965.7	100.0	505.6	100.0	818.9	100.0	931.6	100.0	987.8	100.0	837.4

Dash (-) indicates not applicable.

Source: Survey of County Highway Officials, Fall 1987.

fact that bridges may be sound but, because of width, are inadequate to accommodate the heavier farm traffic of recent years. Also, there are numerous bridges less than 20 ft long, which are not on the Federal Highway System. These structures must be financed by local governments with their own resources.

On the other hand, 31.2 percent of bridges on the NBI were assigned SR values of <50. These bridges require serious attention and, overall, are cause for concern, although not necessarily at the state or federal level. If the bridges serve local traffic only, the argument can be made for strictly local financing, as is often the case.

Bridges requiring attention are not evenly distributed among states. In Missouri, 75.9 percent of county bridges are in the poorest condition category, compared with only 13 percent in neighboring Illinois. Relatively poor states such as Alabama, Arkansas, and Mississippi reported large proportions of bridges needing repairs or replacement. At the same time, Texas and Oklahoma reported 70.1 and 68.3 percent, respectively, of county bridges in the poorest condition category.

TABLE 2 BRIDGES BY SUFFICIENCY RATING AND HBRRP FUNDS RECEIVED

Region and State	Bridges Sufficiency Rating			County Bridges[1]	HBRRP Funds Received		
	Less than 50	50-80	Greater than 80		County	Town/ Township	State
				(Percent)			
Northeast							
Connecticut	-	-	-	0.1	-	-	100
Maine	-	-	-	-	-	25	75
Maryland	37.7	36.5	25.8	26.8	17	-	72
Massachusetts	-	-	-	7.3	-	-	100
New Hampshire	-	-	-	0.1	-	-	-
New Jersey	34.2	36.2	29.6	38.7	-	-	-
New York	50.3	37.2	12.5	35.6	54	-	46
Pennsylvania	42.6	24.0	33.4	12.8	5	3	90
Rhode Island	-	-	-	-	-	-	100
Vermont	-	-	-	-	-	24	75
Mean	***41.2***	***33.5***	***25.3***	***11.0***	***6.9***	***4.7***	***59.8***
Midwest							
Illinois	13.0	37.7	49.3	14.2	10	14	75
Indiana	36.0	26.4	37.6	68.2	33	-	67
Iowa	29.4	42.4	28.2	82.0	47	-	45
Kansas	40.3	37.6	22.1	80.5	23	-	55
Michigan	39.1	20.5	40.4	51.7	75	-	-
Minnesota	24.8	27.5	47.7	67.7	25	-	55
Missouri	75.9	19.8	4.3	63.2	25	-	75
Nebraska	55.1	23.8	21.1	81.6	35	-	50
North Dakota	43.9	23.0	33.1	81.8	30	22	9
Ohio	25.1	35.5	39.4	63.0	15	5	65
South Dakota	43.6	33.0	23.4	73.5	50	-	50
Wisconsin	21.0	39.5	39.5	22.5	9	28	60
Mean	***37.3***	***30.6***	***32.2***	***62.5***	***31.4***	***5.8***	***50.5***
Appalachian							
Kentucky	55.0	33.0	12.0	39.3	15	-	85
Tennessee	50.3	31.5	18.2	49.6	25	-	65
Mean	***52.6***	***32.2***	***15.1***	***17.8***	***8***	-	***30***
Southern							
Alabama	60.4	33.5	6.1	63.2	15	-	80
Arkansas	63.3	29.0	7.7	49.1	15	-	70
Florida	30.1	42.1	27.8	32.0	17	-	83
Georgia	53.0	33.6	13.4	52.7	10	-	84
Louisiana	55.5	32.2	12.3	43.4	15	-	85
Mississippi	60.4	21.3	18.3	72.9	-	-	0
Oklahoma	68.3	19.9	11.8	73.8	40	-	50
South Carolina	48.6	37.0	14.4	12.2	15	-	85
Texas	70.1	21.3	8.6	34.9	13	-	80
Mean	***56.6***	***30.0***	***13.4***	***48.2***	***15.6***	-	***68.6***
Mountain States							
Arizona	21.1	25.6	53.3	19.2	20	-	65
Colorado	55.8	18.0	26.2	48.2	21	-	65
Idaho	20.0	24.6	55.4	38.1	35	-	65
Montana	28.5	38.8	32.7	51.5	30	-	60
Nevada	24.3	37.9	37.8	14.3	35	-	65
New Mexico	31.5	33.0	35.5	18.1	-	-	100
Utah	33.1	31.3	35.6	25.7	45	-	30
Wyoming	45.0	34.6	20.4	30.0	35	-	65
Mean	***32.4***	***30.5***	***37.1***	***30.6***	***27.6***	-	***64.4***
Pacific States							
Alaska	-	-	-	1.8	-	-	100
California	24.2	34.3	41.5	34.3	40	-	50
Hawaii	30.2	28.3	41.5	36.0	15	-	85
Oregon	10.5	31.4	58.1	56.1	25	-	65
Washington	9.6	41.2	49.2	52.3	-	-	-
Mean	***18.6***	***33.8***	***47.6***	***36.1***	***16***	-	***60***

[1] Percent of bridges maintained by counties. Dash:, not applicable.

Source: U.S. Department of Transportation, Federal Highway Administration, ***National Bridge Inventory***, computer printout, 1987; and American Association of State Highway and Transportation Officials, ***Local Finance Survey***, (Washington, D.C.: AASHTO, 1987).

One explanation for Texas and Oklahoma is the adverse economic conditions in the oil industry during the 1980s. State and local revenues became tighter, perhaps forcing local officials to choose between capital and current projects.

Federal bridge dollars are allocated to states on the basis of proportion of total bridge dollar deficiencies within a state. However, internally, funds are allocated by project and various other criteria (*6*). A federal mandate requires that between 15 and 35 percent of the bridge dollars be spent on off-system bridges. State governments also supplement or replace federal bridge dollars with state funds. Thus, the fact that county-maintained bridges in some states are in relatively poor condition is not entirely because of insufficient local resources.

States with a higher proportion of bridge dollars spent on county bridges might be expected to have a smaller proportion of bridges needing major rehabilitation and replacement. This relationship, of course, depends on the percentage of bridges under county control. Table 2 generally supports this hypothesis. In Missouri, where counties receive 25 percent of HBRRP funds, 75.9 percent of the bridges have SR values of <50. Alabama counties receive 15 percent of the HBRRP, and 60.4 percent of the county bridges have SR values of ≤ 50. Arkansas is similar—15 percent of HBRRP funds and 63.3 percent in the poorest category. The allocation of dollars and selection of bridges to receive attention is subject to the state decision-making process. Less state financial support for bridges is reflected in poorer bridge quality in some states.

However, the converse relationship between bridge dollars and condition is not found in all states. Illinois has one of the smallest percentages of county bridges with SR values <50, but counties receive only 10 percent of the HBRRP funding. Illinois, however, has a state bridge program that supplements the federal dollars. Oregon also has a small percentage of bridges qualifying for rehabilitation or replacement, but counties receive 25 percent of the federal bridge support coming into the state.

It is important to make the point that states have varied in response to changing socioeconomic conditions. Fewer miles and fewer bridges are needed today than 50 years ago because of improvements in transportation methods. Closing roads or bridges is not politically popular, and as a result, local officials are tempted to spread financial resources over too many miles, given current traffic needs. Detailed information on this issue is difficult to obtain, but local officials apparently consider reduction in road mileage to be feasible in areas with population declines (*7*, Table 4-1).

HOW COUNTY ROADS AND BRIDGES ARE FINANCED

Nationwide, county roads and bridges are financed with a variety of revenue instruments. Real estate property taxes, as a source of local funds for county highway revenue, are common in 35 states (*6*). State intergovernmental assistance also is important in most states and often involves a sharing of state taxes on motor fuels. Federal intergovernmental assistance through the federal aid secondary highway program, community development block grants, and general revenue sharing is an important revenue source for local road and bridge programs. Federal general revenue sharing was discontinued beginning in 1987, eliminating a readily available source of funding for county roads and bridges.

Revenue Structure

Local governments, depending on state statutes and constitutional restrictions, rely on several revenue sources. The per-mile revenues generated from each source and the proportion that each represents of total revenues are used in this analysis to illustrate the collections and the relative importance of each. Comparisons between 1977 and 1987, the most recent data available, show how these revenues have changed during the past decade.

Per-mile local government receipts increased 7.4 percent in constant dollars between 1977 and 1987, from an average of $2,419 per mile at the beginning of the period to $2,598 per mile in 1987. Two issues are especially important in this comparison: the adequacy of these revenues and their source. Spending need is nearly impossible to determine at this level of aggregation because it depends on type of service, traffic demand, and other factors for which data are not available.

The distribution of the revenue sources clearly has changed, with a greater burden resting on local revenue sources, including user revenues and imposts. Property taxes remained a relatively constant funding source for roads and bridges during the past decade. In 1977, for instance, property taxes represented 18.9 percent of revenues generated by counties and townships for roads and bridges (Table 3). By 1987, the percentage had increased slightly to 19.5 percent, even though during this period many states tightened property tax limits.

A per-mile comparison of property tax collections reveals an overall increase in constant dollars between 1977 and 1987 (10.9 percent) (*3*). This is true even though the unit cost increases for highway maintenance and operation were 102.5 percent between 1977 and 1987. The highway unit cost index is higher than the overall price increases for state and local government purchases, as measured by the gross national product (GNP) implicit price deflator, indicating that agencies responsible for roads and bridges were more adversely affected by cost increases than other agencies (*8*, Table C-3).

Between 1977 and 1982, the growth in real property tax collections was limited by a major recession, high inflation, and property tax limits in many states. However, between 1982 and 1987, a 6.3 percent increase in constant dollars was reported. Thus, for the decade, property taxes increased approximately one percentage point each year and maintained their relative importance, providing approximately 1 of every 5 dollars spent by counties and towns or townships for roads and bridges.

Reliance on combined federal and state intergovernmental assistance declined from 49.1 percent in 1977 to 39.8 percent in 1987. Within the federal aid highway program, the federal aid secondary program assists counties (counties have responsibility for approximately 50 percent of the mileage). Additional federal funds spent on highways during the 1980s include general revenue sharing, community development block grants, and similar programs. Federal assistance to local governments, which provided 9.8 percent of revenues available for roads and bridges in 1977, by 1987 had declined to 5.7 percent. In current dollars per mile, federal aid went from $238 in 1977

TABLE 3 COUNTY AND TOWN OR TOWNSHIP HIGHWAY REVENUE TRENDS

Revenue Source	1977	1982	1987	Pct Change 1977-87
		(dollars)		
Local Receipts per mile				
Current	1,043	1,820	2,800	168.5
Constant[1]	1,043	1,135	1,382	32.5
Property Tax per mile				
Current	457	763	1,027	124.7
Constant	457	477	507	10.9
General Fund Appropriations per mile				
Current	415	620	1,053	153.7
Constant	415	387	520	25.3
User Charges and Imposts				
Current	171	437	720	321.1
Constant	171	273	356	108.2
Federal Highway Aid per mile				
Current	238	300	301	26.5
Constant	238	187	149	-37.4
State Highway Aid per mile				
Current	948	1,314	1,783	88.1
Constant	948	821	880	-7.2
Total Receipts per mile				
Current	2,419	2,690	5,262	117.5
Constant	2,419	2,306	2,598	7.4
		(Percent)		
Reliance on Local Revenue	43.1	49.3	53.2	
Property Taxes	18.9	20.7	19.5	
General Fund Appropriations	17.1	16.8	20.0	
User Revenues and Imposts	2.5	4.3	8.0	
Miscellaneous	4.5	7.6	5.7	
Reliance on Intergovernmental Aid	49.1	44.0	39.8	
Federal	9.8	8.1	5.7	
State	39.2	35.6	33.9	
Reliance on Bonds and Notes	7.7	6.7	7.0	
Total Revenues	100.0	100.0	100.0	

[1] Constant figures are calculated using the FHWA maintenance and operation price index. (See FHWA, ***Highway Statistics 1988***), Table PT-5, p. 60 (1977=100; 1982=160.04; 1987=202.53).

Source: U.S. Federal Highway Administration, ***Highway Statistics 1977, 1978, 1982, 1983, 1987, 1988***, (Washington, DC: U.S. Department of Transportation, Federal Highway Administration), Tables M-1, LF-1, and HM-10.

to $301 in 1987, but in 1977 dollars the 1987 revenues purchased only $149, a decrease in spending power of 37.4 percent.

As a percentage of total revenues, state highway aid is more important than federal aid, and was 39.2 percent in 1977. By 1987, state assistance had decreased to 33.9 percent of the total. This decrease occurred even though many states increased motor fuel tax rates during this period. Nevertheless, state aid remains the single most important revenue source for financing off-system roads and bridges.

In per-mile current dollars, state highway aid was $948 in 1977, and increased to $1,783 in 1987, a growth of 88.1 percent. In purchasing power, though, the 1987 amount was $880, a 7.2 percent decline. In 1977, bonds and notes generated 7.7 percent of the revenues available for roads and bridges, and by 1987 had decreased slightly to 7 percent.

If property taxes remained relatively constant and federal and state support decreased during the past 10 years, where did the revenues arise to finance the roads and bridges? Based on *Highway Statistics* (*3*) data, local governments relied more heavily on user fees and imposts. In 1977, user fees and imposts represented 2.5 percent of revenues collected, but by 1987 they had increased to 8.0 percent.

Fees place more of the burden on road users. Recent surveys of taxpayer preferences conducted by the Advisory Commission on Intergovernmental Relations have shown a pref-

erence for such charges as a method of financing. Detailed information on local user charges and imposts is limited, but at least conceptually this source of revenue is an alternative for replacing shrinking federal and state sources.

Borrowing, another suitable alternative for financing local road and bridge construction, is less appropriate for maintenance, depending on the project. Long-term projects that benefit future generations can be financed with bonds, but short-term projects are less appropriate. The appropriate level of financing from borrowing for roads and bridges is not clear and depends on interest rates. In the late 1980s, interest rates decreased and reduced borrowing costs. The increase in the relative importance of borrowing that occurred in the mid-1980s slowed by 1987.

A more detailed comparison of revenue sources used in financing off-system roads and bridges by region shows widely divergent patterns of financing (Table 4). The Northeast region depends most heavily on property taxes, either earmarked

TABLE 4 RECEIPTS OF COUNTIES AND TOWNSHIPS FOR HIGHWAYS, BY REGION, 1987

Region and State	Local Revenue			Intergovernmental		Bonds	Total Receipts
	Property	General	Misc. Local	State	Federal		
				(Percent)			
Northeast							
Connecticut	-	70.2	1.6	23.9	-	4.3	100.0
Maine	-	72.5	-	23.6	3.7	0.3	100.0
Maryland	3.8	41.4	3.3	39.3	1.8	10.5	100.0
Massachusetts	60.6	-	0.7	32.6	1.6	4.0	100.0
New Hampshire	-	39.2	44.9	15.4	0.5	-	100.0
New Jersey	-	56.6	9.9	3.4	3.0	27.0	100.0
New York	16.2	62.7	2.8	16.0	2.4	-	100.0
Pennsylvania	11.8	38.6	10.8	29.1	9.8	-	100.0
Rhode Island	-	73.8	-	4.0	22.2	-	100.0
Vermont	57.2	2.3	-	31.0	4.6	4.8	100.0
Mean[1]	***13.6***	***51.2***	***5.6***	***22.2***	***3.7***	***3.7***	***100.0***
Midwest							
Illinois	39.0	7.8	21.1	31.1	1.0	-	100.0
Indiana	11.5	0.4	9.3	76.5	2.1	0.3	100.0
Iowa	40.4	0.9	0.6	55.3	2.8	-	100.0
Kansas	50.3	16.6	8.0	21.2	1.4	2.5	100.0
Michigan	1.6	35.4	9.6	49.5	3.8	-	100.0
Minnesota	32.8	7.2	3.6	49.1	6.6	0.8	100.0
Missouri	29.7	5.1	27.6	21.5	7.2	5.8	100.0
Nebraska	43.0	4.9	2.8	32.1	17.1	0.1	100.0
North Dakota	32.0	11.7	2.5	45.7	7.4	0.5	100.0
Ohio	14.3	11.4	5.4	63.7	2.0	2.8	100.0
South Dakota	22.5	36.5	2.8	34.7	3.6	-	100.0
Wisconsin	44.5	24.9	0.2	24.9	0.3	5.3	100.0
Mean	***27.4***	***14.7***	***8.9***	***43.7***	***3.7***	***1.5***	***100.0***
Appalachian							
Kentucky	1.2	12.9	1.4	73.9	8.3	2.3	100.0
Tennessee	16.9	-	6.7	73.4	0.3	2.6	100.0
Virginia	0.1	47.1	40.0	6.5	0.2	6.0	100.0
Mean	***6.5***	***22.0***	***18.9***	***46.6***	***2.1***	***3.9***	***100.0***
Southern							
Alabama	19.3	1.7	5.6	68.7	0.6	4.1	100.0
Arkansas	6.1	15.8	8.3	59.3	10.5	-	100.0
Florida	19.4	15.3	36.5	26.4	0.7	1.7	100.0
Georgia	3.1	50.7	37.0	2.8	0.9	5.4	100.0
Louisiana	22.7	1.1	35.7	30.8	8.6	1.1	100.0
Mississippi	23.1	-	3.1	44.0	19.9	9.8	100.0
Oklahoma	0.4	14.5	2.7	77.9	3.4	0.4	100.0
South Carolina	55.3	-	-	33.4	11.3	-	100.0
Texas	30.6	5.2	13.2	7.1	0.5	43.3	100.0
Mean	***20.8***	***13.7***	***20.8***	***24.3***	***2.7***	***17.7***	***100.0***
Mountain States							
Arizona	2.7	3.2	45.8	46.7	1.6	-	100.0
Colorado	37.7	6.1	6.1	45.2	3.6	1.1	100.0
Idaho	32.3	11.3	5.0	42.9	8.4	-	100.0
Montana	-	-	81.7	11.9	6.4	-	100.0
Nevada	4.3	5.9	16.7	7.7	0.2	56.0	100.0
New Mexico	0.9	10.5	5.7	57.4	2.6	22.8	100.0
Utah	-	47.8	2.8	42.6	6.1	-	100.0
Wyoming	26.2	-	38.2	27.4	8.2	-	100.0
Mean	***14.3***	***7.7***	***25.6***	***36.6***	***3.4***	***10.7***	***100.0***
Pacific States							
Alaska	61.7	18.5	2.1	17.5	0.1	-	100.0
California	1.9	11.4	19.4	44.8	22.5	-	100.0
Hawaii	-	2.0	27.3	60.1	10.6	-	100.0
Oregon	8.0	-	4.7	31.2	46.9	9.1	100.0
Washington	40.2	5.2	4.3	35.8	12.4	2.2	100.0
Mean	***12.4***	***8.5***	***14.0***	***40.8***	***22.7***	***1.7***	***100.0***

[1] Dividing by total number of states in region. Dash (-) indicates not applicable.

Source: U.S. Federal Highway Administration, ***Highway Statistics 1988***, (Washington, DC: U.S. Department of Transportation, Federal Highway Administration), Table LF-1.

for transportation or from general appropriations, with 64.8 percent of revenues derived from this source in 1987. The Midwest region was second at 42.1 percent.

The Midwest region and Appalachian states rely most heavily on intergovernmental aid, a revenue source that represents 47.4 and 48.7 percent, respectively, to support road expenditures. Therefore, shifts in federal or state financing of local roads and bridges can be assumed to affect these regions most.

Substantial variations are evident within each group of states. Alaska, Massachusetts, and Vermont, for instance, rely heavily on property taxes earmarked for roads and bridges. More than three-fourths of the revenues is obtained from state assistance in Indiana (76.5 percent) and in Oklahoma (77.9 percent). Federal aid is especially important in Oregon (46.9 percent), California (22.5 percent), and Rhode Island (22.2 percent).

The widest differences in revenue patterns are in reliance on bonds. Nevada receives 56 percent of road and bridge revenues from this source, followed by 43.3 percent in Texas, and 27 percent in New Jersey. However, differences in reliance on bonds most likely reflect large construction programs within these states, so the percentages may not reflect long-term patterns.

Nationwide, local user charges and imposts provide a growing share of the local support for roads and bridges. Florida and Georgia clearly lead the nation in user charge or local impost usage at the local levels; and, in states such as Arizona, California, Louisiana, Missouri, and Montana, these revenues exceed or at least rival property taxes in relative support of local roads and bridges.

Revenue Outlook

County highway officials reported concern over the revenue outlook, in light of the shifting emphasis toward greater reliance on local revenues to finance roads and bridges (Figure 1). Nationwide, 63.8 percent of county respondents reported inadequate funds, with 43.5 percent reporting that although funds were inadequate for county needs, services were not

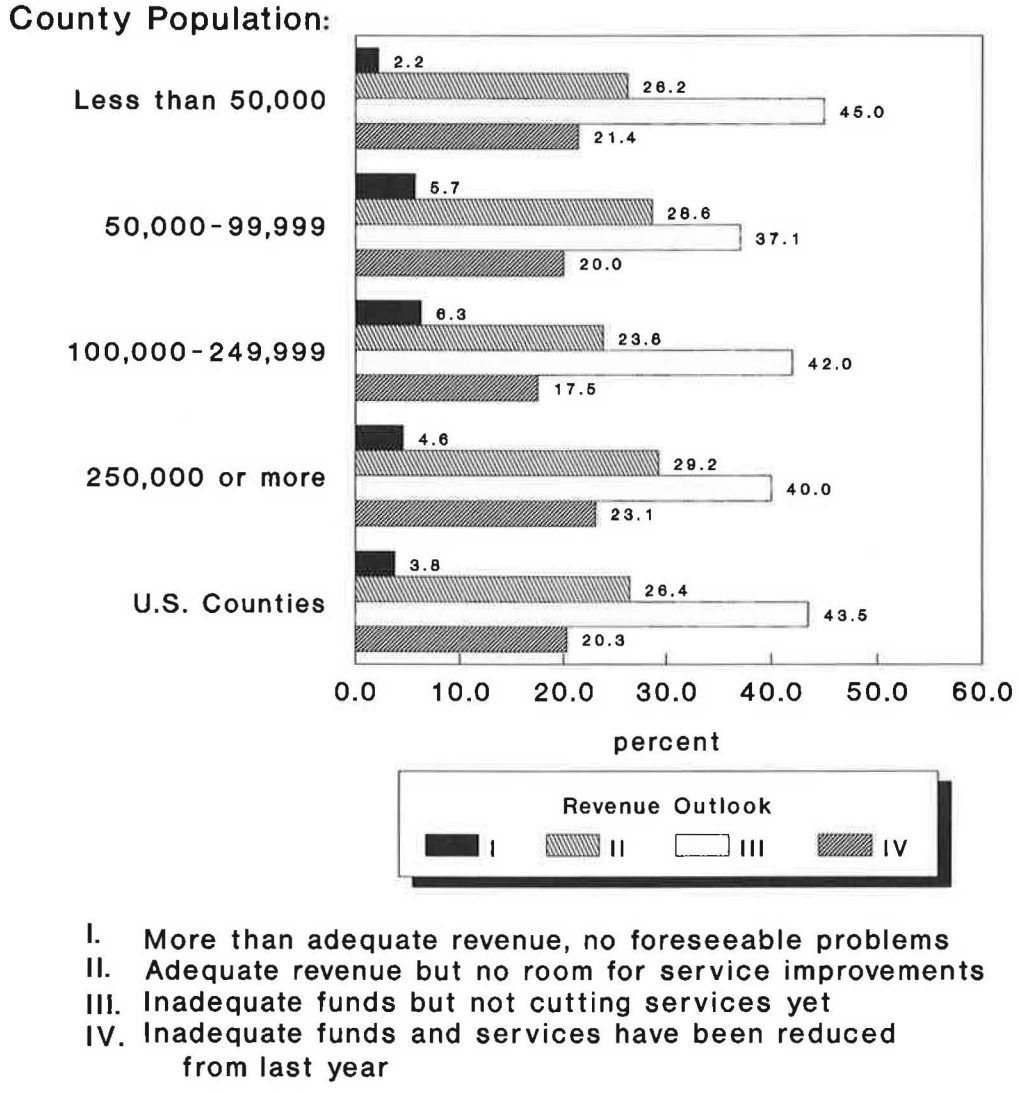

FIGURE 1 County revenue outlook for roads and bridges. (Source: NAC survey of county highway officials, 1987).

TABLE 5 COUNTY AND TOWN OR TOWNSHIP HIGHWAY EXPENDITURE TRENDS

Expenditures	1977	1982	1987	Pct Change 1977-87
		(dollars)		
Capital Outlay per mile				
Current	586	945	1,434	144.7
Constant[1]	586	590	708	20.8
Construction per mile				
Current	516	834	1,255	143.2
Constant	516	521	620	20.2
Maintenance per mile				
Current	1,177	1,918	2,406	104.4
Constant	1,177	1,198	1,188	.9
Roads and Bridges per mile				
Current	1,079	1,766	2,244	108.0
Constant	1,079	1,103	1,108	2.6
Total Expenditure per mile				
Current	2,071	3,602	5,263	154.1
Constant	2,071	2,251	2,599	25.5
		(Percent)		
Capital Outlay	28.3	26.2	27.2	
Maintenance				
Roads & Bridges	56.9	53.3	45.7	
Miscellaneous[2]	14.8	20.5	27.1	
Total Expenditures	100.0	100.0	100.0	

[1] Constant figures are calculated using the FHWA maintenance and operation price index (1977=100, 1982=160.04, 1987=202.53). See FHWA, *Highway Statistics 1988*, Table PT-5, p. 60.
[2] Includes traffic police, administration, interest payment, retirement, and transfer payments.

Source: U.S. Federal Highway Administration, *Highway Statistics 1977, 1978, 1982, 1983, 1987, 1988*, (Washington, D.C.: U.S. Department of Transportation, Federal Highway Administration), Tables LF-21, LF-2, HM-10.

reduced. Of counties responding, 20.3 percent reported inadequate funds and a reduction in services from the previous year. Even allowing for an incentive to overstate financing needs, the response indicates a potential concern. This finding compares with approximately one-quarter (26.4 percent) reporting adequate revenues, but with no room for service improvements.

A careful examination of financial outlook showed relatively small differences in fiscal condition among counties, by size. Counties smaller than 50,000 in population seemed most adversely affected, with 66.4 percent reporting inadequate funds, compared with an average of 63.1 percent for counties of population 250,000 and larger. Nevertheless, the differences are not large and this issue warrants further examination.

The poorest revenue outlook is found in the Pacific states, where 75 percent of counties responding reported that funds are inadequate. The Pacific region also reported roads in the poorest condition. The Midwest followed closely with 73.1 percent in this category. Appalachian and Northeast counties reported the best fiscal position, with 43.4 and 42.9 percent, respectively, reporting adequate revenues.

Expenditure Patterns

Current and constant spending for local roads and bridges in 1977, 1982, and 1987 are compared in Table 5. Special attention is paid to current and capital spending to determine how infrastructure maintenance and upkeep changed during the recession in the early 1980s.

Previously, a majority of local expenditures were for maintenance, but in recent years, they have shifted to capital expenditures. Nationwide, in 1977 local governments spent an average of 56.9 percent on maintenance, but the spending decreased to 45.7 percent in 1987. Capital spending was relatively stable, decreasing slightly from 28.3 percent in 1977 to 27.2 percent in 1987. The dependence on maintenance and

capital spending between 1977 and 1982 shifted to miscellaneous spending. In constant dollars, capital and maintenance spending both increased slightly during that period. Between 1982 and 1987, capital outlays increased slightly in proportion, whereas maintenance declined substantially. The miscellaneous category, including traffic police, administration, interest, debt, retirement, and transfer payments, increased from 14.8 to 27.1 percent between 1977 and 1987. This increase probably was caused partially by increases in interest rates.

An overall increase in real per-mile capital expenditures for roads and bridges occurred between 1977 and 1987 (Table 5). Local governments spent an average of $586 in 1977, and by 1987 had increased to $708 per mile for capital outlays. Between 1977 and 1982, the growth was negligible, whereas a substantial increase occurred between 1982 and 1987. Construction expenditures increased from an average of $516 per mile in 1977 to $620 per mile a decade later.

When the maintenance and operating unit cost index is used to depict price changes, maintenance expenditures between 1977 and 1987 are flat, actually declining slightly in constant dollars between 1982 and 1987. This finding is interesting. A likely scenario is that the high inflation and economic recession in the early 1980s limited expenditures for roads and bridges and the constant dollar increases were small. Demand created during this period may have pressured local officials to catch up on construction projects at the cost of maintenance expenditures.

With the available data, determining factors associated either with the condition of roads and bridges or with the ability to finance them, however defined, is difficult. However, several variables were collected as part of the NAC national survey that can shed some light on perceptions of local highway officials about budgets, conditions, and related issues.

CORRELATES OF ROAD CONDITION

Simple correlations were run between road condition (percentage of roads rated inadequate in the county), whether a county faced property tax limitations (1 is no; 2 is yes), mileage per capita in a county, per capita expenditures, dependence on property taxes, whether a county is metropolitan area or rural, budget outlook (Table 4), and per capita income. The correlations are presented in Table 6.

Two variables are related to perceived road condition: whether a county faces property tax limitations and budget outlook. Counties with effective limitations also are those in which road conditions were rated lower ($r = -0.1609$). This finding may indicate that counties with poorer quality of roads have had to levy relatively higher property taxes to make improvements and therefore are approaching property tax limits ($r = -0.1927$). However, the condition and proportion of general revenue coming from property taxes are not correlated ($r = 0.0958$).

A statistically significant correlation exists between spending per mile and property tax limits ($r = -0.1705$). Thus, counties without property tax limits spend more on average. The quality measure, it must be remembered, is a rating by local highway officials based on perceived needs, not an arbitrary standard based on projected traffic demand.

Equally interesting is the funding that road condition is not significantly related to per-mile expenditures ($r = 0.1024$) or number of miles per capital under county responsibility ($r = 0.0693$). This finding suggests that local officials in counties in which spending is higher do not necessarily view the road conditions as better. Counties with higher spending may have demands for greater traffic or better quality of roads. However, counties with higher per-mile spending had significantly

TABLE 6 CORRELATION MATRIX

	Road Condition	Property Tax Limit	Miles per Capita	Highway Exp per Mile	Property Tax/ General Revenue	Metro/ Nonmetro	Budget Outlook	Per Capita Income
Road Condition	1.000							
Property Tax Limit (1=no; 2=yes)	-.1690[1]	1.000						
Miles per Capita	.0693	.0527	1.000					
Highway Expenditure per Mile	.1024	-.1705[1]	-.2591[2]	1.000				
Property Tax/General Revenue	.0958	-.1927[1]	.2485[2]	-.1052	1.000			
Metro/Nonmetro	.0245	-.0218	-.3391[2]	.3995[2]	-.0773	1.000		
Budget Outlook	-.3885[2]	.2582[2]	.0129	-.1202	.0033	.0439	1.000	
Per Capita Income	.0873	-.0067	-.2260[2]	.7126[2]	-.0807	.5420[2]	.0521	1.000

[1] Significant at 1 percent. [2] Significant at .1 percent.

Source: Survey of County Highway Officials, Fall 1987.

higher per capita incomes ($r = 0.7126$). Higher per-mile spending also is associated with fewer miles per capita ($r = -0.2591$).

Counties in which roads were rated in poorer condition also reported a bleaker budget outlook ($r = -0.3885$). These are counties (budget outlook) with property tax rate limits ($r = 0.2582$), suggesting that local officials may feel pressed for additional revenue sources. State-imposed property rate tax limits could be raised, if deemed necessary, to alleviate the budget outlook problem.

These findings suggest that there are counties in which local officials perceive road conditions are not sufficient for local needs, but they already rely on property taxes for financing. Although these counties have not spent significantly less or more than other counties, they perceive that the budget outlook is not particularly bright. The perceived budget outlook is not related to per capita income (a measure of wealth) or whether the county is metropolitan or rural.

OPTIONS AVAILABLE TO LOCAL GOVERNMENTS

This analysis is not intended to make policy recommendations, but local governments have had to respond to real declines in federal and state support of local roads and bridges. Depending on statutory and constitutional authority, local governments can respond in several ways. Not all of the following options are equally suitable in each state but most are feasible and local policy makers should at least consider them.

Increased Property Taxes

Low-volume roads and bridges, especially in rural areas, predominantly serve local constituencies and therefore by necessity are financing primarily with local resources. In rural areas, this financing can present a major difficulty because of sparse populations and the relatively high costs per resident of financing roads. A broad industrial base to shoulder the property tax burden often does not exist. In the late 1970s and early 1980s, with the poor farm economy, differential property tax assessment programs were instituted in many states. The differential assessment limited property tax revenues available for roads and other local services. Increasing property taxes may involve raising state statutory limits.

Reduced Public Road Mileage

Existing road networks were created under a much slower transportation system. Changes in agricultural marketing methods and transportation improvements have reduced the road mileage in some cases so much that a road is no longer feasible. This point is generally recognized and has been documented in previous studies (*7*). Although reducing road mileage makes sense conceptually, politically this policy is difficult to implement because adjoining landowners lose access to roads but gain little in property tax savings. In addition, state motor fuel tax reimbursement programs sometimes use per mile as a base. Reducing mileage means less revenue, even though infrequently traveled roads may not receive a proportionate expenditure.

The access limitation issue can be partly solved by converting the road to private use, and there is a clear indication that this has happened. Residents can maintain the road as they see fit or can convert it to agricultural use. In either event, the local government no longer has responsibility, but it may lose the state aid associated with this mileage.

Population declines, especially in rural counties, are a fact of life, and services, including roads and bridges, are affected. When traffic volume reaches a point that the mileage is no longer financially feasible, provisions must be made to reduce the mileage. This option, although not popular, will be increasingly necessary in rural states.

Limited-Maintenance Roads

As a compromise to closing roads, some efforts have been mounted to stipulate that certain low-volume mileage will receive limited maintenance. When the mileage is clearly marked so that travelers bear the risk, local governments no longer have legal liability for accidents. Local governments may receive the same state funding but can allocate their budgets to higher-volume mileage. Over time, residents may change their travel patterns so that the mileage will eventually be closed.

User Charges and Imposts

Instituting user charges may require state authorizing legislation. User charges also may be difficult to implement in rural areas with low-volume roads, because there are relatively few travelers to charge. Imposing costs on developers or heavy users can discourage economic development. An example in which user charges and other financing arrangements have been effective is extractive industries such as coal mines. Agreements with mine owners to rebuild local roads or to pay for a permit to use the roads have been effective in some areas. Even though user charges and imposts have gained popularity during the past decade, they remain at less than 10 percent of the revenues available and are concentrated in relatively few states, such as Arizona, California, and Florida. Certainly, other states could review, and possibly learn from, these experiences.

Reorganized Local Road Network

Local and state policy makers could benefit from rethinking the local highway networks. Considering certain mileage as consisting of high-quality access roads and relegating other mileage to other uses can reduce the overall costs. Pennsylvania has found this approach to be effective; other states could possibly examine Pennsylvania's experiences.

CONCLUSIONS

Low-volume roads and bridges are financed mainly by counties, towns, or townships. These governments, especially in

rural areas, experienced economic setbacks during the 1980s from declines or slow growth in tax revenues, as well as from loss of federal general revenue sharing. Although spending on local roads and bridges has increased, the role of local resources also has increased.

During the late 1970s and 1980s, counties and townships underwent two distinct trends. Between 1977 and 1982, they reduced capital projects in favor of current operations. After the recession, local governments spent relatively more on capital projects, presumably to make up for earlier delays.

The condition of the low-volume roads and bridges, as perceived by local public officials, is viewed with some concern. More than 1 mi in 5 (22.9 percent) of the county-maintained roads are considered inadequate for traffic, and 35.6 percent are rated as only barely adequate or worse. A similar situation exists with county-maintained bridges of 20 ft or longer. In states such as Missouri, more than three-fourths of the county bridges are in poor condition.

What also must be recognized is that population declines in rural counties may mean that some mileage is no longer economically feasible. Local officials must realize this fact and take action to bring the road mileage in line, not only with the ability to finance, but also with traffic volumes. These adjustments will not be made as long as state distribution systems are based on road mileage that is maintained.

The revenue outlook for counties, in the minds of responding highway officials, is not bright. Nationwide, about two-thirds of county highway superintendents responding to a mail survey said that existing funds were inadequate, and 20.3 percent reported service cutbacks. Little on the horizon suggests that conditions will become much better.

Local infrastructure is essential to the future development of rural areas. Rural residents rely on roads and bridges for essential public services as well as for gainful employment. Declining populations will make the financing of low-volume roads and bridges more difficult in rural counties. Yet without continued attention, the transportation system may deteriorate further.

Addressing the rural transportation financing issue will not be easy. A host of societal issues from basic services to water quality and waste management face local governments. Roads and bridges are but one of the issues that must be addressed. Local officials must evaluate the need for low-volume roads and bridges and, if they seek continued federal support, must make the case that rural roads are of critical importance to the national highway system. State governments must evaluate policies that may prevent effective local responses to change. The federal government must determine the role it will play in financing off-system roads and bridges. Each ultimately plays a part in the overall solution.

ACKNOWLEDGMENT

The authors thank Poh P'ng, Carol Rachus, Nancy Baird, and Diane Pollock for assistance in data tabulation and manuscript preparation. The research for this paper was financed through a grant with USDA, Office of Transportation, while McWilliams was Chief, Office of Rural Economics and Policy.

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Financing of Low-Volume Roads in Argentina

CASTOR LOPEZ

Considering the wide spectrum of problems that needs to be addressed in the administration of road systems in developing countries, the financing of low-volume roads in Argentina deserves particular attention. In dealing with the overall problem of improving the efficiency in the assignment of resources within the public sector of the Argentine economy, an analysis of the economic objectives of road financing is presented through a proposal leading to the implementation of a road pricing mechanism. One purpose is to present a cost structure model of the Argentine road system to identify the optimal objective that would lead to an efficient and equitable system of road financing. This analysis would make it possible to channel private resources to the maintenance of the road network that the country needs, as well as to return the concession of public works to a private sector as an instrument available for the financing of roads in Argentina. On the other hand, this same analysis can be used to compare road system costs and revenues by type of vehicle and surface of road, from which it can be deduced that the present system of road user fees in Argentina is producing an important hidden cross-subsidization from low-volume to primary-road users. This process, in turn, causes the overutilization of paved roads and the underutilization of rural—generally nonpaved—low-volume roads. This inefficiency in the assignment of the resources for roads calls for a profound revision of the present revenue system, which is unjust to the users. It is necessary to turn to a taxing policy that would function like an incentive—pricing to marginal costs, for example—for a more rational utilization of the roads in the country.

"There are no free roads in the world, there are just different ways of paying for them."—William J. Miller, Jr., United States.

In view of problems that the growth of road systems has given rise to in developing countries, special attention is needed to the financing of road management and maintenance. For example, in Argentina motor vehicle registrations have increased six times in the last 25 years without a similar increase in road maintenance sources.

The experience in the evolution of road transport in Argentina since 1960—responsible in the short term for most passenger and goods transport—has clearly shown that it is much more difficult to develop an institutional capacity for proper road maintenance than to build the road networks themselves.

In Argentina during the last 10 years at least, roads have been built but not maintained. That situation is important in this country, where rural production makes intensive use of roads: 90 percent of grain transport and almost 100 percent of livestock transport are by trucks.

This research, which purports to be a needs analysis and a pricing study, presents an analysis of the costs that Argentine road agencies should expend to maintain properly not only rural and secondary roads, which are usually gravel or earth roads, but also paved interurban roads. Likewise, these costs of road infrastructure are broken up among the various users according to their utilization of the roads (*1*).

In the same way, this provides an analysis of the revenues that road organizations should receive as specific resources; it implies dedication of user fees. These taxes for the utilization of roads, or an indirect toll, represented by charges on different vehicle inputs, are also broken up among the users (*2*).

Interesting information arises from the comparison of these road costs and the user taxation hypothesis, identified by type of vehicle and road. Analysis shows that the present road tax system produces an unfair participation of the users in the financing of the various types of roads and thus generates a clearly inefficient resource allocation.

The main inputs on which road taxes are imposed—gasoline, oils, and tires—do not appropriately reflect the road consumption of the users, making these charges invalid as road prices or inefficient as assigners of road resources. This would produce an important hidden cross subsidy from gravel and earth road users to paved road users, giving rise to a detrimental effect of overuse of high-volume roads and underuse of rural low-volume roads.

In Argentina, the average daily traffic (ADT) on primary paved roads is 1,650 vehicles per day (vpd), on gravel roads it is over 100 vpd, and on earth roads it is almost 50 vpd. The study focuses primarily on the interurban network and on the inequity between road user charges and the actual costs of different classes of vehicles on that network. However, there are also inequities in the allocation of costs caused by urban traffic congestion and by the combination of all user taxes into a central, single fund. The incorrect allocation of resources based on the current processes has impaired the maintenance of the primary network and has resulted in a critical need to rebuild 10 000 km of primary roads.

The realization of serious inefficiency in the allocation of road resources, the relevant unfairness toward low-volume road users, and the recent program for the reconversion of the road network should lead to a profound conceptual revision of the road resources collecting system. This should result in a criterion of user taxation on marginal road costs and a more equitable cost-allocation policy because it would promote the idea of user taxation and rational utilization of the entire road network (*3*).

Finally, within the present context of restructure of the role of the public authorities in the road transport sector, it is considered that this scheme of analysis establishes conceptual

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foundations (*4*) to favor the canalization of resources from the private sector towards the road infrastructure that the country imperatively needs in the short term, and to place the concession of public work (*5,6*) and road toll back into the frame of available means (*7,8*) for the financing of the maintenance and improvement of roads in Argentina.

COSTS OF ARGENTINE ROAD AGENCIES

The treatment of the costs of Argentine road agencies follows the research approach developed by authors (*9–11*) who support the concept of user taxation on the basis of marginal costs of road use.

This concept shows that taxes should be charged on the economic costs of user trips and not on the costs of the construction or maintenance of the facility. Neither of these costs can be reduced when road users do not use the road. With the aim of achieving efficient allocation of the existing and available road resources, it would be a mistake to charge them at a fraction of the costs of the trips of the road system users.

The costs of the road system that have to be paid by the users vary with the volume of traffic. This concept of collection—known as the "marginalist" economic criterion—leads to the maximization of the net social benefits and ensures an efficient allocation of the scanty productive goods (*12,13*), but it also implies revenues lower than expenditures in the budgets.

To keep the concept of the research and the character of the methodology of the model used, the following simple relevant assumptions are made:

1. The initial assumption made is that the land transport sector is in a state described in economics as "first best" (*14*), and the criterion is that road transport—both passengers and goods—should neither take up nor generate resources in relation to other means of transport and to the rest of the economy.
2. The costs of the roads to any agency are related to the management, construction, maintenance, and right of way. They are classified as variable and invariable—whether they depend on the traffic or not—to allow the treatment of the costs of road organizations by means of an economic analysis, in accordance with the traditional approach of the theory of the firm.
3. The differentiation of the construction, rehabilitation, and maintenance costs into invariable and variable is a highly problematic matter. A criterion—life cycle cost—based on the observed periodicity in the investments during a road's useful life was used.
4. Although the climate and topography are highly important variables in determining road costs, at this stage of the study the temporal and regional variability of road costs are not considered, assuming that such costs depend only on transit through the concept of the ADT.
5. At first, this transit is seen through the criterion of a hypothetic standard vehicle, and then it is assumed that there are four types of vehicles: automobile, bus, and light and heavy truck. Likewise, the model considers three categories of roads: paved, gravel, and earth or dust, thus shaping a 3×4 matrix on which the analysis will be based.

In order to determine the costs of the road agencies in Argentina, it was necessary to process not only their expenditures as a result of the contracts with the private sector to execute the rebuilding, improvement, and maintenance works—through price tenders and private and public biddings—but also to estimate the costs of routine maintenance works, generally carried out under the supervision of the road agencies themselves.

The concept of marginal cost is defined as "long-term marginal costs," so it includes as variables the short-term fixed costs. Starting from the generally accepted expression of costs C of any road agency, with their components (fixed A and variable B) in terms of transit ADT,

$$C\ (\$/\text{km-year}) = A\ (\$/\text{km-year}) + B\ (\$/\text{ADT/km-year}) * \text{ADT} \quad (1)$$

From the analysis and statistical treatment of correlations of the investments done in the different categories of roads (paved, gravel, and earth) as well as the transit assessed on the same, the estimated values of the annual fixed road agency Cost A and annual variable road agency Cost B are presented in Table 1.

By replacing the values obtained in the general expression of agency road costs, the following system of equations is inferred for 1989:

$$C_{\text{pavement}}\ (\$/\text{km-year}) = \$1{,}800.00 + \$1.25 * \text{ADT} \quad (2)$$

$$C_{\text{gravel}}\ (\$/\text{km-year}) = \$350.00 + \$4.15 * \text{ADT} \quad (3)$$

$$C_{\text{earth}}\ (\$/\text{km-year}) = \$150.00 + \$10.00 * \text{ADT} \quad (4)$$

By equating the expressions, it is possible to estimate the break-even ADT values when the agency road costs of maintenance become equal for two categories of roads. Strictly, for initiating improvements like paving a gravel road, it has to be determined by equating life cycle costs, including user costs, for the two categories of roads. In fact, the break-even values (ADT 500 and 34) coincide approximately with those obtained in similar analyses in other Latin-American countries (*15*), and they would validate the results and their cost relations.

The variable agency road costs detected have been estimated for the hypothetic standard automobile referred to, also called "equivalent." According to the analysis of the range of vehicles using the Argentine road network, users can

TABLE 1 ESTIMATE OF COSTS OF ARGENTINE ROAD AGENCIES

Road Category	Invariable Cost A (US$/km-year)	Variable Cost B (US$/km-year-ADT)
Paved road	1 800.00	1.25
Gravel road	350.00	4.15
Dust road	150.00	10.00

NOTE: Invariable costs include all the road investments that do not change with the transit in the short term. Variable costs are the road agency's expenditures that change with the traffic stream.

be grouped in the four categories mentioned: automobiles and pickups, buses, light trucks (without trailer), and heavy trucks (semitrailer and trailer).

In order to determine the incidence of each of these categories of vehicles in the costs of road agencies, there are basically two possible concepts to be used: A fundamentally geometric criterion, called road space occupation (*16*), based on mechanical and operational characteristics of the vehicles, and other structural criterion, or road consumption, which takes into account mainly the destructive effect of loads through axles on the road (*17*).

In the present analysis, a mix of both concepts is chosen, predominating the incidence of the structural criterion on paved roads and that of road space occupation on gravel and earth roads, resulting in costs allocated to each type of users over each road category as presented in Table 2.

It is appropriate to emphasize that, for automobiles and buses, the marginal costs of road agencies are increasing from paved roads to natural soil roads; conversely, for light and heavy trucks, marginal costs are decreasing in this sense, although they are ever higher than formerly. This can be explained by the mainly superficial consequences of light transit on gravel and dust roads and by the effects on lower layers of heavy transport, which affects pavements to a greater extent.

A large portion of ADT consists of automobiles, and the cost per kilometer of operation declines in dust roads, which are cheap to build, deteriorate rapidly with use, and must be bladed often. Light and heavy trucks do about the same damage as a car on dust roads, but they are a small portion of the vehicle stream, so the damage done per passage is less expensive to repair than the damage to paved roads.

The estimative character of the long-term marginal costs allocated to the various users should be especially considered, in view of the simplifying assumptions that have to be done to modelize the cost structure of Argentina's road agencies, as well as numerous conflicts that have been overcome by means of hypotheses and solutions of agreements, always questionable. However, this information, under reasonable observation, is valid.

PRICES FOR ROAD UTILIZATION

The particular characteristics of road networks must be considered in relation to financing (*18,19*), especially the temporal variability of the concept of roads as public goods. When traffic volumes are lower than the highway capacity, the assumptions of nonrivality and nonexclusion are valid regarding road space consumption among the users and the financing can continue to be done under the concept of road use tax or indirect toll. These roads with very low volume to capacity ratios are noncompensatory.

But, in spite of the fact that even congested roads are quasi-public goods, when roads approach their capacities, producing congestion, these assumptions need to be discussed. Crowding out among users appears and it's possible to establish a proposal for fixing road prices by means of direct tolls, whether open or closed, to ration efficiently the scanty factor road and oblige the transit to internalize all the negative externalities associated with traffic congestion. In this case, the revenues will exceed agency direct costs.

All that conforms to a public service system in which it is of prime importance for the private costs of use of the different vehicle users on the different road categories to be as similar as possible to the costs society has to afford—through road agencies budgets—to provide each of them with a road infrastructure service.

It's fundamental to perceive this optimum allocation—ideal but valid as an objective to aim at—because the private costs of operating the various vehicle users are the ones that developed the role of prices for road using. Consequently, the correct assessment of the costs ensures an approximation to an efficient allocation of the resource road equally available among the users.

In this sense, the operating costs of the four types of vehicles on the three road categories are analyzed according to usual methodologies (*20*). Because the average operation speed is the fundamental variable to determine the vehicle costs, it was necessary to assume average values observed in Argentina's roads. The results are presented in Table 3.

With the operating speeds defined for each vehicle-road situation, estimated road taxes were broken up, starting from the total operation costs, to compare this revenue with the variable costs B; such charges were identified as taxes on gasoline, oil, and tires, which were added to the taxes on the ownership of vehicles such as for license plates and on buying and selling. Such components of the user cost were assumed as road price or indirect tariff because of road use, having the values presented in Table 4 as a result.

TABLE 2 ESTIMATE OF VARIABLE COST B OF ROAD AGENCIES (US$ 1989/km-year-ADT)

	Cost (US$) by Road Category		
Type of Vehicle	Pavement	Gravel	Natural Soil
Car	1.25	4.15	10.00
Bus	7.19	14.53	18.58
Light truck	55.48	22.83	19.80
Heavy truck	93.83	39.43	24.20

NOTE: The ADT is decomposed in each road category using PCEs and ESALs. These equivalencies were obtained with the assumption of different mix of both criteria for the three road categories on the basis of average operation condition, which may vary significantly for the four given vehicle types.

TABLE 3 AVERAGE OPERATING SPEED ON ARGENTINE ROAD NETWORK

	Speed (km/hr) by Road Category		
Type of vehicle	Pavement	Gravel	Natural Soil
Car	105	85	60
Bus	90	75	50
Light truck	85	65	40
Heavy truck	70	55	40

NOTE: The research needs speed to find user revenue contributions because some of the costs subject to user taxes—fuel consumption, tires, and oil do vary with operating speed to the same degree. All the vehicle variable operating costs are some function of speed.

TABLE 4 USER REVENUE CONTRIBUTIONS

Type of Vehicle	Contribution (US$ 1989/km-year-ADT) by Road Category		
	Pavement	Gravel	Natural Soil
Car	7.22	8.01	9.14
Bus	11.74	13.83	15.62
Light truck	17.62	21.32	28.99
Heavy truck	26.44	37.73	44.71

NOTE: These user revenue contributions were obtained from the user taxes and the assumptions of equivalencies.

HYPOTHESIS ON VEHICLE COMPOSITION AND TRAFFIC DISTRIBUTION IN ARGENTINA'S ROAD NETWORKS

In order to start with the comparative study of the present financing situation of low-volume rural roads with gravel or natural soil surfaces in contrast with paved roadways in the country—treatment made through the cross-subsidization theories—it is necessary to determine the scope in which the model is developed.

Although it makes up more than 600 000 km of roads (*22*,*23*), the vast country road network was not taken into account in the analysis mainly because of the little existing information regarding the type and characteristics of the surfaces and the volume of transits.

The study was carried out on the arterial or primary road network (paved) and secondary systems (generally nonpaved), road infrastructure that includes as a whole the distribution according to road category as presented in Table 5.

Various vehicle composition and ADT values were assumed on this network in each case, according to measurements and correlations. The ADT values assumed for each road category are the product of the measurements (*24*) and are in accordance with the range of transits found in the analysis of costs of Argentina's road agencies, as presented in Table 6.

Table 4 and indirect tolls indicate that, for all kinds of vehicles, charges always increase from paved roads to natural soil roads. This is in accordance with marginal costs, which also increase (Table 2) because of automobiles and buses.

The marginal costs of light and heavy trucks increase more radically as these vehicles use lower-grade roads. Comparing user revenue contributions (Table 4), user revenue for cars increases 26 percent for vehicle use of soil roads compared with paved roads. Light trucks increase 64 percent and heavy trucks increase 69 percent. Since gravel roads and natural soil roads are the first stage of transport of Argentine rural production, it is important to correct or reduce this failure in the present criterion of road fund taxes on fuels or gasolines, oils, and tires, that impacts the space farm—paved roadway.

TABLE 5 ANALYZED ARGENTINE ROAD NETWORK

Road Category	Length (km)	Percentage
Pavement	55 000	26
Gravel	40 000	19
Natural soil	115 000	55
Total	210 000	100

SOME RESULTS

In a balanced road-financing system (*25*,*26*), such deficiencies because of positive gaps between revenues and marginal costs are necessarily compensated by revenues lower than marginal costs on other types of users, through the mechanisms developed in the cross-subsidization theories.

Although strict comparisons between the marginal costs of road agencies—broken up according to vehicle and road type—and the corresponding indirect tolls, also classified according to user and road category, would provide important data for analyzing responsibilities in investments and assumed tariffs, the variety of taxes in road transport in Argentina and their permanent and sharp relative variations make it necessary to pay close attention to the measurements in order to obtain information that in any event will generally have a short period of validity.

From the marginal costs of road agencies (Table 2) and the indirect tolls detected (Table 4), under the hypothesis of utilization assumed in the system of analyzed roads, it can be inferred that if the whole of the specific resources that conform to the revenues of road agencies are taken into account, they would overpass by about +147 percent their present needs.

But, because less than 20 percent of the specific impositions go to road funds, that strong surplus changes into road revenues that are only enough to cover 53 percent of the long-term marginal costs, thus causing the present state of damage of the Argentine road network.

As a consequence, the research work was developed under the assumption of a balanced road budget that allows carrying out a detailed analysis of each type of vehicle and road category, determining their percentage shares both in the expenditures and in the revenues of the road system by means of a

TABLE 6 PERCENTAGE OF VEHICLE COMPOSITION

Type of Vehicle	Road Category		
	Pavement (ADT = 500 vpd)	Gravel (ADT = 120 vpd)	Natural Soil (ADT = 30 vpd)
Car	60	70	75
Bus	5	7	10
Light truck	15	13	10
Heavy truck	20	10	5
Total	100	100	100

TABLE 7 ARGENTINE ROAD COMPOSITION EXPENDITURES (1), REVENUES (2), AND COVERAGE OR EQUITY RATIOS (3)

Type of Vehicle	Road Category			
	Pavement	Gravel	Natural Soil	Total
Car	(1) 2.37%	1.60%	3.04%	7.01%
	(2) 25.84%	5.83%	5.24%	36.91%
	(3) 10.90	3.64	1.72	5.27
Bus	(1) 1.14%	0.53%	0.74%	2.41%
	(2) 3.50%	0.96%	1.17%	5.63%
	(3) 3.07	1.81	1.58	2.34
Light truck	(1) 26.30%	1.68%	0.79%	28.77%
	(2) 15.75%	2.96%	2.17%	20.88%
	(3) 0.60	1.76	2.75	0.73
Heavy truck	(1) 59.32%	2.18%	0.31%	61.81%
	(2) 31.54%	3.93%	1.11%	36.58%
	(3) 0.53	1.80	3.58	0.59
Total	(1) 89.13%	5.99%	4.88%	100.00%
	(2) 76.63%	13.68%	9.69%	100.00%
	(3) 0.86	2.28	1.99	1.00

NOTE: When the direct road system marginal cost recovery or equity ratio (3) is <1, it indicates underpayment of revenue relative to marginal cost responsibility.

coverage ratio, as presented in Table 7. An analysis of both vehicle type and road category follows.

Analysis According to Vehicle Type

Automobiles and Pickups

This type of vehicle on paved roads is heavily taxed; its contributions surpass the costs they cause to road agencies approximately 990 percent. On rural gravel roadways and dusty country roads, the situation is similar, and the contributions surpass the expenditures 260 and 70 percent, respectively. On the whole, the taxes of this category of vehicle surpass its costs more than 400 percent, and the equity ratio shows that overall passenger cars pay 5.27 times their marginal cost responsibility.

Buses

This type of vehicle also covers its costs, surpassing them 200, 80, and 58 percent on pavements, gravel, and dust roads, respectively. On the whole, its contributions surpass 130 percent of the expenditures caused to road organizations.

Light Trucks

On pavements, this type of vehicle only covers 60 percent of its expenditures, whereas on gravel and natural soil roads its contributions exceed the costs by 76 and 175 percent, respectively. In general, its contributions reach 73 percent of expenditures.

Heavy Trucks

In this case, the subsidy on pavements becomes more acute, the revenues are only 53 percent of its costs, whereas on gravel and dust roads, the contribution surpasses the expenditures by 80 and 258 percent, respectively. In view of the strong incidence of its high traffic volumes on paved roads, it covers on the whole only 59 percent of the costs produced to the agencies.

Analysis According to Road Category

Pavements

Taking into account the conditions of system length, vehicle composition, and assumed traffic, for this type of road, with 26 percent of the network under study, 24.5 million vehicle-km circulate daily (77 percent of total traffic), and in spite of the relevant surplus obtained from automobiles and buses, the existing subsidies for trucks cause the revenues only to reach 86 percent of the investment in this road type.

Rural Gravel Roads

In this system, which accounts for 19 percent of the total analyzed network with a circulation of about 5 million vehicle-km daily (13 percent of the total traffic), the users, in all vehicle categories, contribute +128 percent of the expenditures of the agencies in this road category.

Country Dust Roads

Three and one-half million vehicle-km daily (10 percent of total traffic) circulate in these 115 000 km, 55 percent of the model network; and the taxes on the users of this system surpass about 100 percent of the costs of the work done by road organizations in this road category.

CONCLUSIONS

Every vehicle type covers its costs at present, except for the case of trucks, both light and heavy, on paved roads; in these

cases the subsidy is of such magnitude that it cannot be compensated either by the positive gaps of all the other types of vehicles on paved roads, or by the trucks themselves in the other road categories.

This situation would cause a clear handicap to the users of low-volume rural and country roads—usually gravel and dust roadways—who, in every vehicle category, easily cover the costs, in contrast to the sector of users of high-volume paved roads that are strongly subsidized.

Such a situation can be described in economic terms indicating that, at present, low-volume rural and country road users contribute 23.37 percent (13.68 and 9.69 percent, respectively) of road revenues and are responsible for 10.87 percent (5.99 and 4.88 percent, respectively) of the total costs of the road system. On the other hand, paved road users contribute the remaining 76.63 percent of the revenues but cause 89.13 percent of road expenditures.

Consequently, under the protection of a distorted policy of road charges, in Argentina at present there are two cross-subsidy effects in operation: (a) from light to heavy road transport and (b) from rural and country low-volume road transit to arterial roads of the main network.

In order to correct simultaneously these important detected subsidies, the possibility of establishing an effective control of vehicle weights and loads on all roads in an interconnective network and applying a toll to trucks circulating on the main paved roads should be considered in the short term.

Such a toll, which should be a function of the destructive effect trucks cause to pavements (*6*), would be assigned to the reactivation of an agricultural roads plan, an interesting experience for rural low-volume roads done in Argentina 15 years ago, with a new financing mechanism, no longer through taxes on agriculture exports, but through tolls for heavy transit on paved roads, as a way of making up for, or at least reducing, the detected subsidy.

It is fundamental to understand that to aim at an optimum utilization of a given road network, it is necessary that each and every user assume the resulting costs of their consumption of roads.

Allocating user fees on the relationships between user costs and consumption of roads is known as marginal cost pricing. Considering other factors such as developmental road programs, subsidy programs, and specific taxes based on nonefficiency does not alter these marginal cost relationships.

It may be necessary for political reasons to allocate resources on a noneconomic efficiency basis. This is because political equity is not the same as the economic Pareto optimality that would be achieved through the perfect actions of a laissez faire economy.

In this sense, it is considered that the recent Enactment Plan of Road Reconversion and the State Reform and Economic Emergency laws, with their required enactments, assume the critical situation of the sector with a criterion of realism, equality, and efficiency.

These tools, simultaneously with the reorganization of the "Direccion Nacional de Vialidad," decentralizing and offering greater participation to provincial road agencies, provide genuine mechanisms to balance, through the initiatives of participation of the private sector in a competitive context, the historical road deficit in Argentina.

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Road Network Investment System for Developing Countries

SAMUEL TURAY AND RALPH HAAS

Funding for road improvements has not been able to keep pace with needs in most, if not all, developing countries. Consequently, it has become more and more necessary to develop systems for managing the road network, including the efficient programming of projects so as to make the best possible use of limited funds available. The development of models to aid investment decisions for the road sector in developing countries began to receive impetus in the late 1960s through work initiated by the World Bank. Today, such comprehensive models as the Highway Design and Maintenance Standards Model (HDM–III) are available. What is needed in addition are management systems for taking over after the basic investment decision has been made so that specific and optimal network rehabilitation and maintenance programs can be produced. A methodology in the form of the Road Network Investment System (RONIS) for establishing road investment priorities is adjustable and suitable to low-income, developing countries. The system was applied to an actual sample network of paved, gravel-surfaced, and earth roads in Sierra Leone, West Africa, to illustrate the following: determination of the present status of the network, identifying improvement needs, developing priority programs for specified funding levels, and analyses to assess the impacts of different funding levels on future serviceability of the network.

In most highway departments, funding for rehabilitation and maintenance has not been able to keep up with the improvement needs. This problem is generally more visible in developing countries. Consequently, cost-effective allocation of shrinking resources in the improvement and rehabilitation of road networks in developing countries has become particularly difficult. The desired result can be accomplished through the development and implementation of road management systems (RMSs).

The application of models to aid investment decisions for the road sector in developing countries received impetus in the late 1960s through work initiated by the International Bank for Reconstruction and Development. Today, such comprehensive models as the World Bank's Highway Design and Maintenance Standards Model (HDM) (*1*) and the Road Transport Investment Model (RTIM) (*2*) are available for this purpose. These models are products of a large amount of experimental and analytical work incorporating four environments: Kenya, Brazil, Caribbean, and India. However, the models are essentially directed to decision making for overall road investments. In their simplest forms, the models are used to minimize the total cost of transportation over the life of either one road link or an entire network.

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A major issue in the use of these systems in developing countries is the lack of a comprehensive priority subsystem as a program and financial planning tool. The primary functions of such a tool include determination of the network status at various points in time, improvement needs, and evaluation of different policies on a short-term but detailed basis within the context of the network level of management.

After the basic investment decision has been supported by the HDM or RTIM models, there is a real need for a system to then take over and produce specific optimal programs and financial planning lists at the network level. In order to detail improvement needs, to group them into viable projects, and to produce a refined budget and program of works, such a system would require performance prediction models that suit the needs of growing databases in developing countries. The current condition of a road will be the key input data for performance prediction.

A simple, practical, and yet comprehensive system for programming and planning network improvements both for unpaved and paved road sectors in low-income developing countries is described. An illustration of the use and general applicability of the procedure is provided through a case application in Sierra Leone, West Africa.

THE ROAD NETWORK INVESTMENT SYSTEM (RONIS)—AN OVERVIEW

RONIS, which is a menu-driven, user-friendly, microcomputer-based software system, incorporates three major modules, as shown in Figure 1.

Input Data and Candidate Analysis Module (ICAM)

The first module, ICAM, starts with inventory of the road sections. These inventory data, involving structural and surface condition, traffic volume, road geometrics, etc., are used to establish the present status of each road section. A road deterioration (performance prediction) model is then applied to predict their remaining service lives. The output of this phase consists of a list of sections that are candidates for improvements.

Economic Analysis Module (ECAM)

The second module, ECAM, concerns detailed economic analysis. It considers those sections that are estimated to have

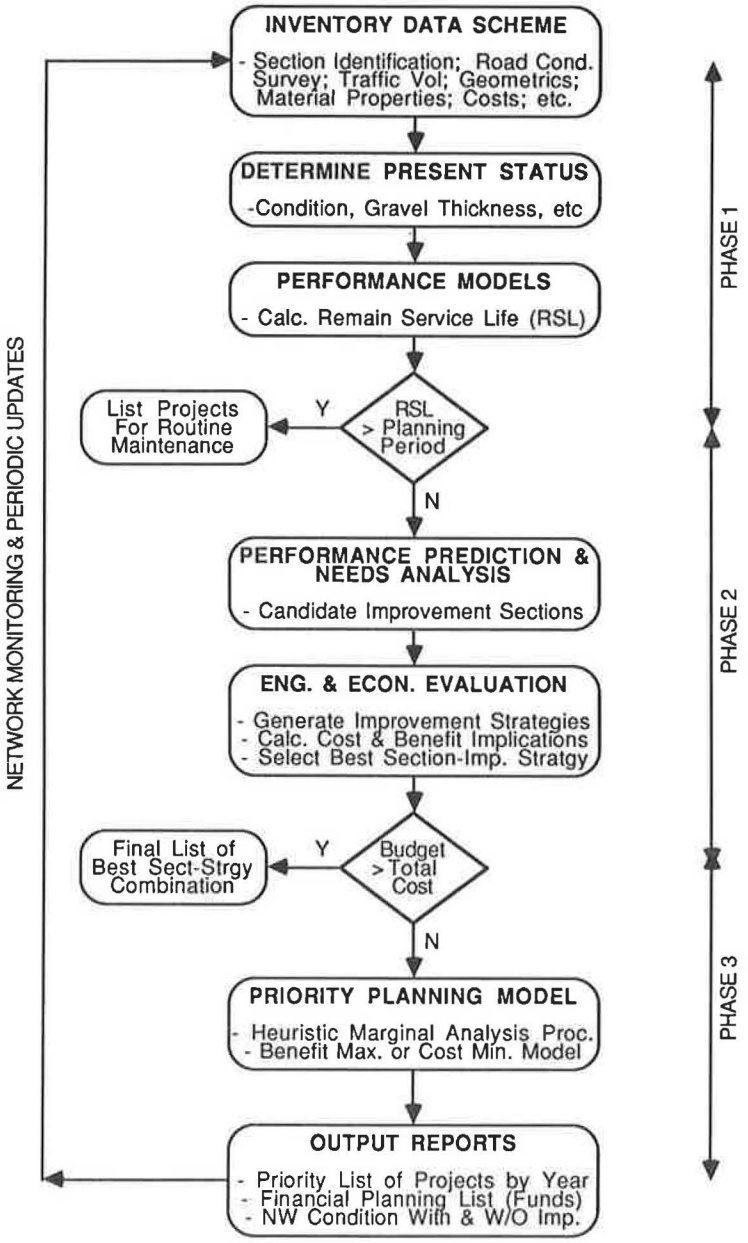

FIGURE 1 General structure of RONIS.

a remaining service life less than the planning period. At the end of the remaining service life (i.e., needs year), several surface improvement strategies are generated. Four strategies for paved roads (do nothing, single overlay, double overlay, and reconstruction) and three strategies for graveling (lifts of 5, 10 and 15 cm) for unpaved roads, and the possibility of paving of unpaved roads, are considered.

Performance is predicted and the economic implications are calculated for each strategy. In other words, costs related to improving the road and benefits (i.e., savings in vehicle operating costs because of the improved road condition) are calculated.

The output of this phase is a list of candidate sections, alternatives and their direct costs to the agency, and user benefits for each possible implementation year in the planning period.

Heuristic Analysis Module (HAM)

The final module, HAM, uses the outputs of ICAM and ECAM as inputs in a priority programming model. HAM is a near-optimization model that establishes priorities over the whole network on the basis of benefit maximization and budget constraints. It also recommends an optimum improvement strategy for each road section considered in the analysis.

The final output of the system is an optimum rehabilitation strategy and implementation year for each project. It is based

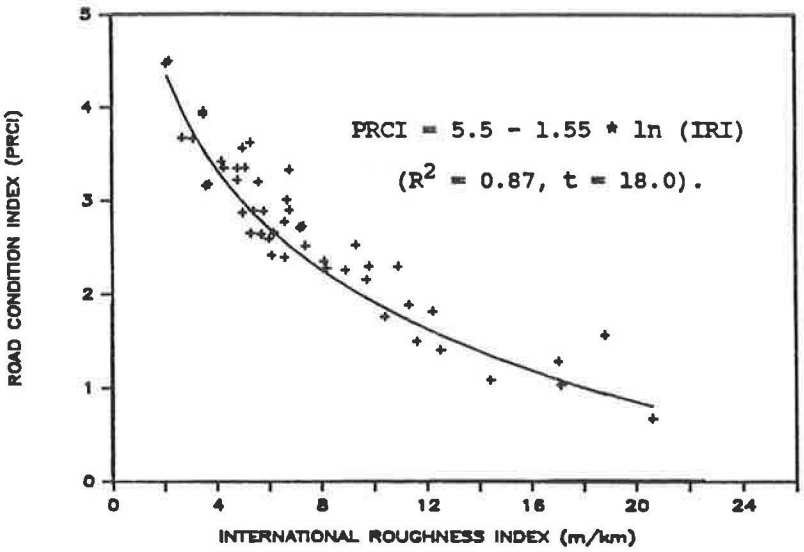

FIGURE 2 Relationship between serviceability and road roughness.

on the maximization of net annual worth (NAW) and at the same time ensures that the agency will stay within its budget in each year throughout the planning period.

RONIS incorporates four major concepts; serviceability of a road, performance prediction models, economic evaluation, and priority analysis, which are further discussed in the following section.

DEVELOPMENT OF A SERVICEABILITY MODEL

The serviceability of a road is an essential element in the development of RMSs. Serviceability is needed to define the present status of road sections in order to locate abnormal changes and deficiencies, and thereby identify improvement needs. In addition, serviceability is required to describe the life history of a road for the evaluation of alternative improvement strategies and to establish a system-wide basis for allocation of road improvement resources.

In the practical use of the serviceability concept, if the roughness of a road section is measured mechanically, the road user's response can be estimated in terms of a serviceability index.

The data base used for developing the serviceability model is taken from Brazil's International Road Roughness Experiment (IRRE) study (*3*). The data matrix consists of 18 raters and 49 road sections varying from earth to asphalt concrete roads. The rating results were analyzed to see if any systematic error was involved in the ratings using the procedures described by Hutchinson (*4*). Several models were hypothesized; the recommended one is shown in Figure 2.

PERFORMANCE PREDICTION MODELS

Performance is a general term for how roads change their condition or serve their intended functions with accumulating use of traffic and environment (i.e., over time). One of the most useful approaches, particularly in developing countries where historical data are limited, involves the development of a model based on the current state of the road.

Performance of Unpaved Roads

The deterioration of unpaved roads is characterized primarily by roughness and gravel loss. Other useful measures, such as rut depth, are not explicitly taken into account because their effects are subsumed in the property of roughness along the wheel paths.

Blading Frequency Model

Road roughness increases with time through the continuous action of traffic and alternating seasonal effects. At high roughness levels, a blading operation is undertaken to smooth the surface and potholes. Hence, a sound blading policy is the most important first step in prolonging the life of unpaved roads. An objective way is to blade the road often enough to ensure levels of roughness that minimize the sum of vehicle operation and blading costs. Data from several sources in developing countries (*5–7*) have been used to develop a regression-based model between blading frequency and daily traffic volume.

Figure 3 indicates a minimum of one blading per year for roads carrying less than 50 vehicles per day; and six to eight bladings for high-traffic volumes. Consequently, although Figure 3 is based on a limited sample size, it can be used as a first estimate in most regions. Because of differences in materials and economic circumstances, caution should be exercised in generalizing these findings without more comprehensive local field studies.

Graveling Operation Model

Predicting the loss of gravel material is important in defining the present status of a road and in planning future graveling

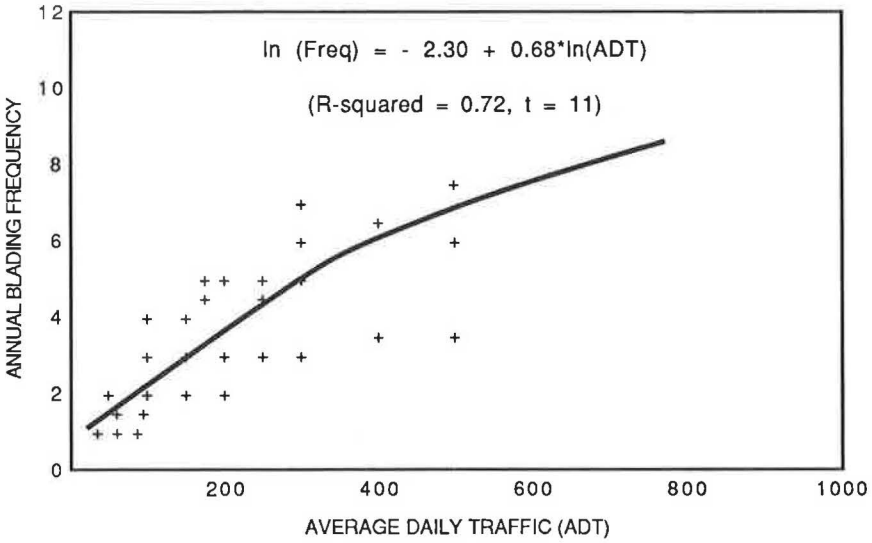

FIGURE 3 The effect of traffic volume on the annual blading frequency.

operations. An agency needs to know when to program such operations for the entire network. A recursive model is formulated of the form

$$G_t = G_0 - k * t \quad (1)$$

where

G_t = Gravel thickness (mm) at any time t,
G_0 = Present gravel thickness (mm),
k = Annual gravel loss (mm), and
t = Period between observations (years).

Visser's gravel loss model (*8*) is used as an initial basis for estimating the k values for different classes of roads. Road sections having similar characteristics (e.g., surface type, traffic, and geometrics) can be grouped into classes. The implicit assumption in the use of road class is that a change in the level of each factor will result in a significant change in the loss of surface material.

Three levels of traffic (low, medium, and high); three levels of terrain (flat, rolling, and steep); and one surface material (lateritic gravel) have been considered, as presented in Table 1. Each class of road is defined by the combination of each of these factors.

The k values in Table 1 are class dependent. Horizontal curvature has a minor effect. Those responsible for graveling want to know the frequency in terms of years rather than months. An average radius of curvature is 400 m, which reflects typical design standards of unpaved roads.

Performance of Paved Roads

A useful approach in predicting pavement performance, particularly when historical data are limited, involves the Markov method. This approach captures the knowledge and experience of local engineers, superintendents, and technicians for predicting changes in road condition with time for various combinations of traffic, layer types and thickness, and subgrade strength. Details of the use of Markov modeling procedures have been described by Karan and Haas (*9*) and by Turay (*10*).

The essential requirements of the Markov chain model include the definition of pavement states, pavement classes, initial state vector, and transition matrices. Subjective opinions of five pavement engineers from Sierra Leone have been used to develop the initial transition matrices for 12 road classes (see Table 2), as described by Turay (*10*). This procedure was used because of unavailability of detailed field inventory data.

The flexibility and ease of the Markov approach make it appropriate for transfer from one country to another, particularly for situations in low-income countries. Figure 4 gives typical performance curves for three of the road classes from Table 2 under a do nothing strategy.

ENGINEERING ECONOMIC ANALYSIS

In most RMSs, detailed cost analyses are necessary for in-depth project evaluation. At the network level, however, average or benchmark cost information may be considered adequate for priority analysis, as shown in Table 3 for Sierra Leone conditions.

User benefits are defined as savings in vehicle operating costs (VOCs) caused by improving the road's condition. The reason for selecting this criterion is two-fold. Technically, major road cost studies have conclusively shown that VOC, which is the largest component of the total life cycle cost of roads in developing countries, is extremely sensitive to variations in road roughness (*1,2,11*).

From a national viewpoint, developing countries have always emphasized the economics of road transport because of the influence of international development agencies, which require that every project be economically justified before lending money to the bank-assisted countries.

Therefore, the introduction of benefits that are defined as savings in VOCs make the evaluation more practical and above

TABLE 1 PERFORMANCE FACTORS AND ANNUAL GRAVEL LOSS VALUES OF CLASSES OF UNPAVED ROADS

CLASS OF ROAD	TRAFFIC VOLUME (ADT)	TYPE OF TERRAIN	ANNUAL GRAVEL LOSS (k), cm		
			RADIUS OF CURVATURE (m)		
			300	400	500
1	Low	Flat	1.4	1.3	1.2
2	Low	Rolling	1.7	1.6	1.5
3	Low	Steep	2.0	1.9	1.8
4	Medium	Flat	1.8	1.7	1.6
5	Medium	Rolling	2.0	1.9	1.8
6	Medium	Steep	2.4	2.3	2.2
7	High	Flat	2.5	2.4	2.2
8	High	Rolling	2.8	2.7	2.6
9	High	Steep	3.2	3.0	2.9

PERFORMANCE FACTOR	DESCRIPTION
Traffic Level (ADT)	Low < 100; 100 < Med. < 300; High > 300.
Traffic Mix	20 % cars; 50 % light vans; 30 % trucks
Terrain Type (%)	Flat < 2; 2 < Rolling < 5; Steep > 5.
Surfacing Material	Laterite Gravel: < 10 P.I., < 20 % pass. 0.074 mm.
Environment/Climate	6 months each - wet and dry seasons;
	Avg. annual temp = 28 deg. C., Rainfall = 200 cm/yr.

all make it possible to compare surface improvement projects with other competing transportation demands.

The Kenya VOC relationships (*11*) were selected because the corresponding VOC components could easily be estimated as a function of road roughness, bypassing all intermediate steps requiring estimation of road geometrics, vehicle dynamics, etc. This is a major advantage because the focus of RONIS is on improving the surface condition of road networks.

The basis for economic evaluation is the NAW method. Of course, the benefits must exceed costs if a project is to be justified on economic grounds. For unlimited funding, the best section and strategy combination is selected. For limited funding, as is always almost the case, a priority subsystem is needed.

PRIORITY PLANNING MODEL

There are two main approaches for tackling the management problem of optimization of resources. The first approach can be summarized as selecting projects in such a way as to maximize the total benefits accrued from improving the road without violating the resource constraints. This approach is strictly a program planning model. The second approach can be summarized as selecting projects in such a way as to minimize road improvement expenditures subject to prespecified performance levels. This approach is generally referred to as the financial planning version of road management.

These two optimization approaches represent the limits from "do the best for the road network" to "do the best for the road-using public." A compromise can be made by selecting a program planning model, and then determining the average condition of the network before and after the investment decisions are made. In effect, this process should take into consideration the importance of both the road user and the effect of the plan on the performance of the road network.

A heuristic, marginal NAW procedure has been used (*10*) that proceeds in three main stages. First, an initial allocation routine assigns candidate road sections to their need years. The routine maintenance budget is deducted from the total budget to allocate the budget required for major surface improvements. Marginal analysis is then applied to select the best improvement strategy for each road section, given the required budget for that year. If the list produced by this allocation routine is feasible within prespecified tolerances of the budget (e.g., ±5 percent), it is the optimal solution.

The second stage, which involves a reallocation routine, displaces candidate road sections by years to satisfy the budget constraints. In this procedure, some road sections may get advanced or delayed. They may even be pushed over the end of the planning period, which in practice means that they will not be improved within the planning period.

TABLE 2 PERFORMANCE FACTORS OF CLASSES OF PAVED ROADS

CLASS OF ROAD	TRAFFIC VOL (AADT)	SURFACE THICKNESS	SUBGRADE STRENGTH
1	High	Thick	Strong
2	High	Thin	Strong
3	High	Thick	Weak
4	High	Thin	Weak
5	Medium	Thick	Strong
6	Medium	Thin	Strong
7	Medium	Thick	Weak
8	Medium	Thin	Weak
9	Low	Thick	Strong
10	Low	Thin	Strong
11	Low	Thick	Weak
12	Low	Thin	Weak

PERFORMANCE FACTOR	DESCRIPTION
Traffic Level (AADT)	Low < 500; 500 < Med < 1500; High > 1500.
Traffic Mix	20 % cars; 50 % light vans; 30 % trucks
Pavement Type	Flexible (Asphalt Concrete).
Surface Thickness (cm)	Thin < 5.0; Thick > 5.0.
Subgrade Type	Strong - Granular (CBR > 12); Weak - Clay.
Environment/Climate	Avg. Temp. = 28 deg. C/yr., Rainfall = 200 cm/yr.

The final stage of the procedure estimates, for each year, the average condition of the network that would result from the implementation of the priority list (including that of zero capital funds or do nothing). This added information is useful in recommending a financial plan on the basis of desired road performance levels.

SAMPLE APPLICATION

The system was applied to an actual sample network in Sierra Leone involving 80 sections consisting of 35 paved roads, 30 gravel roads, and 15 earth roads, totaling about 700 km (center line). Detailed input data for each section [e.g., location,

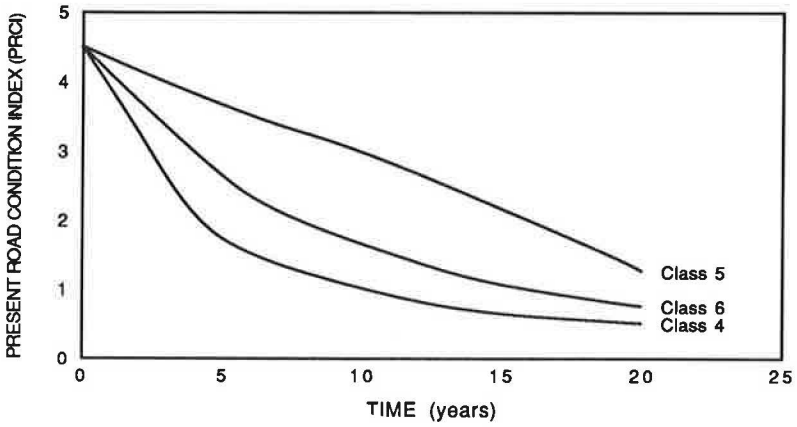

FIGURE 4 Typical performance curves under a do nothing policy.

TABLE 3 TYPICAL BENCHMARK ROAD IMPROVEMENT COSTS

UNPAVED ROADS		PAVED ROADS	
Activity	Cost ($/km)	Activity	Cost ($/lane km)
Light Blading	150	Single Overlay (Resealing)	12,000
Heavy Blading	300		
Graveling	9/cu. meter	Double Overlay (Strengthening)	19,000
Paving	24,000		
		Reconstruction (Restoration)	50,000

length, annual average daily traffic (AADT), present road condition index (PRCI), roughness, last improvement strategy, lane width, subgrade type, grade, and gravel or surface thickness have been provided by Turay (*10*)].

In order to conduct the analysis, the following assumptions were used:

1. A minimum acceptable PRCI level of 2.0 was used for paved roads. Thus, a section that had a PRCI of 2.0 or less was identified as requiring a major improvement in the year.
2. A critical thickness of 7.5 cm for gravel roads was used in the analysis. A bidirectional, threshold traffic volume of 75 vehicles per day (vpd) is used as a screening device for upgrading (or graveling) an earth road.
3. A road section is considered in the optimization process starting only from the year in which it first reaches its terminal level. Once the road reaches its terminal level, it is kept at that level with a penalty cost until major improvement is scheduled.
4. A planning period of 5 years was selected to truly represent the National Development Plans of most developing countries. Also, a discount rate of 10 percent is used.

Present Status of Network

Figure 5 shows the distribution of the key performance parameter defining the status of each road sector. For the total sample network of 700 km (center line), the paved sector represents about 45 percent and has a mean PRCI level of 3.0; gravel roads constitute 35 percent with a mean thickness of 10 cm; and earth roads constitute 20 percent with a mean bidirectional traffic volume of 60 vpd. In these examples, all mean values are weighted by the total length for each surface type.

Improvement Needs

The needs year is the point in time at which the performance curve for a particular road section reaches its terminal acceptable level. Figure 6 shows the distribution of needs for the example network over a 5-year planning period.

Under a do nothing policy, a total of 63 out of 80 sections become needs for the 5-year plan. In Year 0 (i.e., present needs), for example, three paved roads representing 4 percent of the entire network; nine gravel roads representing 10 percent of the network; and two earth roads representing 3 percent of the network are expected to become needs.

Priority Planning Results

The needs list in the previous section would have been the priority list if unlimited funds were available. However, under budgetary constraints, as is always almost the case in developing countries, some projects cannot be built in the year in which they require improvements. They may have to be delayed in time or not scheduled for rehabilitation at all, depending on the funds available.

Priority analyses were carried out for several budget levels. Table 4 presents a typical detailed, section-by-section list (for a budget level of $3.25 million annually) that results from the analyses. Figures 7 and 8 show a summary distribution of the recommended priority list for a budget level of $3.25 million per year for a 5-year period. Spending priorities are given to routine maintenance before allocating extra funds to major improvements activities.

The reconstruction strategy does not show up in the priority list because all projects having a negative value of NAW (i.e., uneconomical feasible projects) are deleted from further priority analysis. This policy, in effect, reinforces the notion that roads should not be allowed to deteriorate into poor condition, which requires far more costly reconstruction and lower marginal benefits compared to other improvement alternatives.

Budget Level Analysis

A summary of the long-term effects of different spending decisions is shown in Figures 9 and 10. For example, an annual budget of $2.5 million would allow the paved-road sector to drop only slightly from its current average PRCI level of 3.0; but nearly 50 percent of these roads would become deficient by the end of the planning period (Figure 9). For the gravel-surfaced roads (see Figure 10), an annual funding level of $2.5 million would allow the average gravel thickness to stay at about 10 cm, and the deficient mileage to drop near zero. However, the sensitivity to funding level is quite apparent in Figure 10 in that a lower annual budget would lead to a dramatic rise in deficient mileage.

For comparative purposes, a spending decision of zero capital budget (i.e., apply only routine maintenance) has been included in Figures 9 and 10 to show its extreme effects on the network as a whole. An additional impact of such an action would be significantly higher user and maintenance costs and the likelihood of losing much of the existing investments (i.e., complete reconstruction would eventually be required).

Several funding levels were also used to test the sensitivity of the system to various decision variables. Inspection of Figure 11a reveals that the number of sections in the priority list increases with increased funding. For example, 16 paved and 32 unpaved sections will be the recommended improvement priorities if a budget level of $2.5 million per year is allocated to the network. At $3.25 million annually, on the other hand, all needs sections (i.e., 31 paved and 32 unpaved sections)

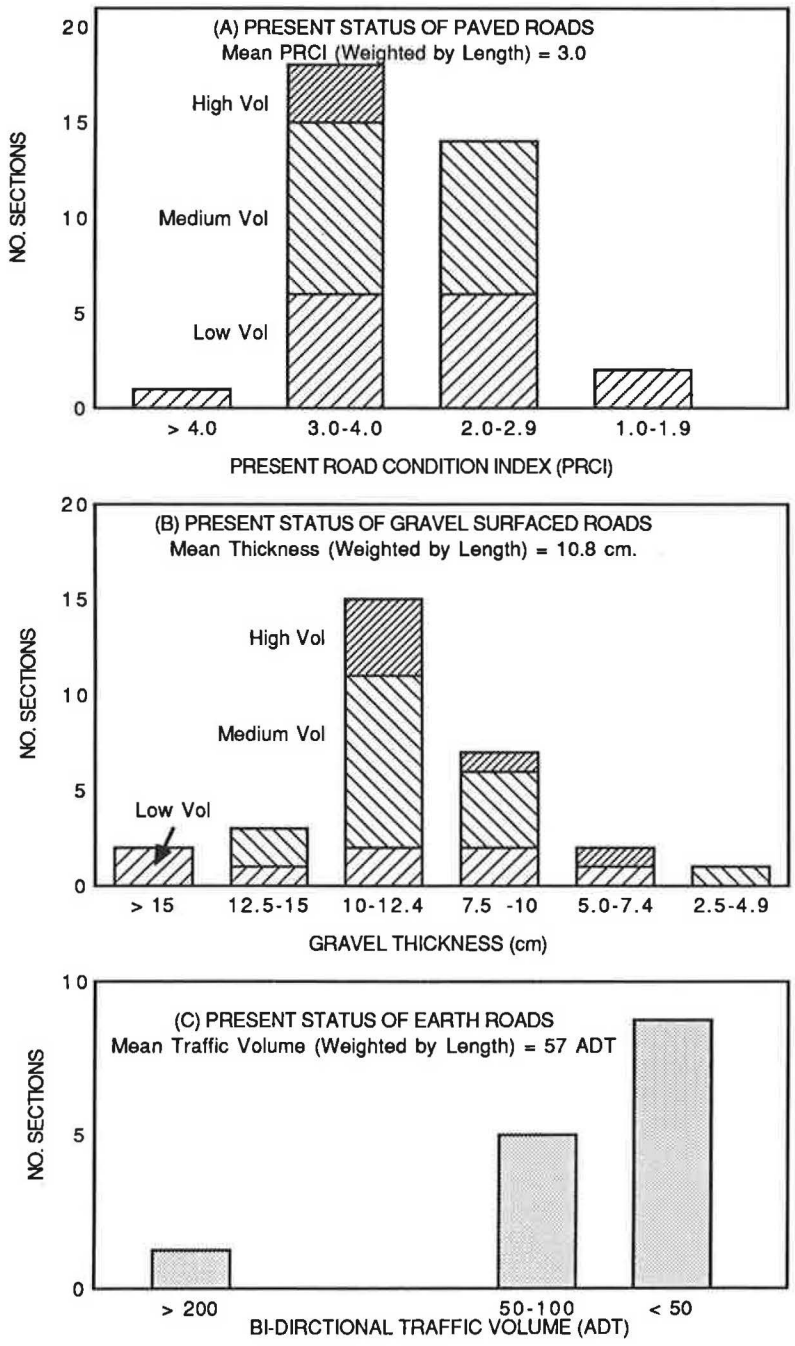

FIGURE 5 Present status distribution for a regional network in Sierra Leone.

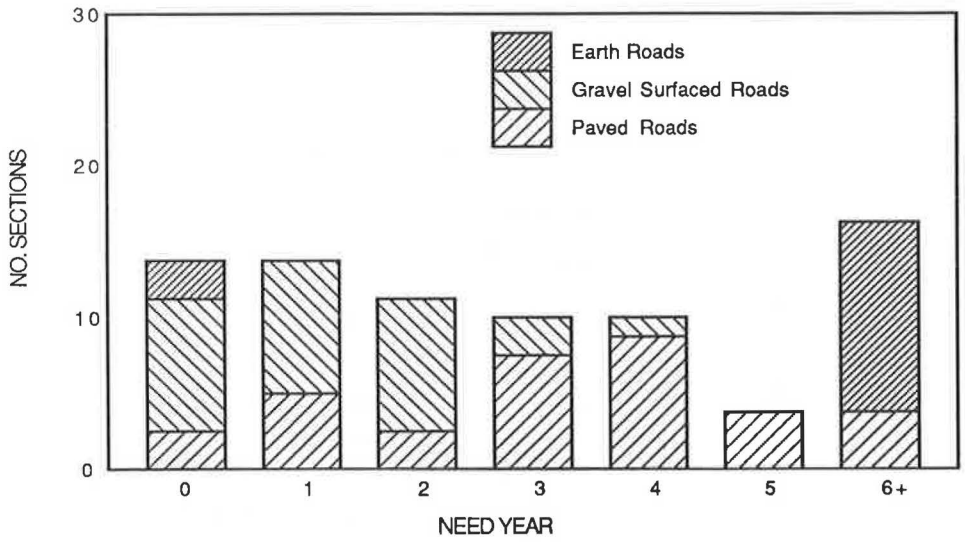

FIGURE 6 Distribution of need sections for the example network.

are considered for improvements. However, at some point, increasing the budget level does not significantly change the priority list. Spending beyond this point would represent an inefficient use of funds.

Figure 11b shows the average network condition at the end of the planning period. The plots reach an asymptotic PRCI level of 3.5 for paved roads and a thickness of 12 cm for the unpaved sector at a funding level of $3.5 million annually. But there is little difference in going from $2.5 to $3.5 million annually. Also, in Figure 11c, the value of the objective function (i.e., NAW) increases substantially up to about $3.0 million annually, and then only slightly after that, to approximate a steady state between annual funding levels of $3.0 and $3.5 million.

The foregoing analysis suggests that the annual funding level should be at least $2.5 million for this network but that $3.0 million annually is a level most advantageous to the sustained performance of the roads, highway department, and the road-using public. This type of analysis provides an objective tool for senior highway administrators to justify their budget requests and to demonstrate the consequences of reduced funding on the average condition of the entire network.

SUMMARY

A management system is presented for evaluating road network improvements in developing countries. It is complimentary to basic investment decision models such as the World Bank's HDM–111. The examples indicate how the concepts of pavement management can be used to provide an objective and systematic means for planning and justifying road improvement expenditures in developing countries.

A heuristic model for optimization of available resources in developing countries is simple and cost-efficient. It can easily be implemented in a microcomputer environment. The procedure is comprehensive in that it uses a marginal analysis approach that yields a near-optimal solution. The limitation of not obtaining a strict global optimality has been traded off against practicality and the efficient use of computer resources.

RONIS can assist developing countries in answering a number of questions, including the following:

1. What is the present condition of the network and what will it become if a particular funding level is selected?
2. What sections need improvement, when is the work going to be scheduled, and how should the work be done?
3. What is the justification of increased funding or the consequences of reduced funding decisions?

RONIS provides sufficient information for determining the optimal funding level as well as establishing a premise for setting performance standards for low funding situations. RONIS allows highway authorities not only to justify a proposed funding plan but also to set funding levels on particular networks.

TABLE 4 A DETAILED PRIORITY LIST FOR AN ANNUAL FUNDING LEVEL OF $3.25 MILLION

RECOMMENDED PRIORITY LIST FOR YEAR 1

ROAD NAME	TYPE	FROM (km)	TO (km)	TRAFFIC VOLUME	IMPROVEMENT POLICY	IMP. COST ($1000)	NEED YEAR	OTHER YR. TO IMPROVE RD.
Hill St. - Hastings	Paved	7.5	15.5	240	Single Overlay	192	1	
	Paved	15.5	24.5	200	Single Overlay	216	1	
Kambia - Madina	Gravel	9.5	18.5	200	5 cm Gravel	25	1	2 (10); 5 (5)
Kambia - Pamlap	Gravel	39.0	45.0	500	5 cm Gravel	18	1	2 (10)*
	Gravel	31.5	39.0	500	5 cm Gravel	23	1	2 (10); 5 (5)
Lungi - Mahera	Paved	17.8	23.3	150	Single Overlay	228	1	
	Paved	27.3	37.7	150	Single Overlay	250	1	

RECOMMENDED PRIORITY LIST FOR YEAR 2

ROAD NAME	TYPE	FROM (km)	TO (km)	TRAFFIC VOLUME	IMPROVEMENT POLICY	IMP. COST ($1000)	NEED YEAR	OTHER YR. TO IMPROVE RD.
Hill St. - Hastings	Paved	24.5	33.5	100	Single Overlay	192	2	
Kambia - Madina	Gravel	0	9.5	200	10 cm Gravel	57	2	
	Gravel	18.5	28.5	120	5 cm Gravel	30	2	5 (5)
	Gravel	28.5	37.0	120	10 cm Gravel	49	2	
	Gravel	37.0	45.0	80	5 cm Gravel	24	2	
Lunsar - Makeni	Paved	0	10.0	1200	Single Overlay	240	2	

RECOMMENDED PRIORITY LIST FOR YEAR 3

ROAD NAME	TYPE	FROM (km)	TO (km)	TRAFFIC VOLUME	IMPROVEMENT POLICY	IMP. COST ($1000)	NEED YEAR	OTHER YR. TO IMPROVE RD.
Lungi - Port Loko	Paved	0	6.4	250	Double Overaly	230	3	
Masiaka - P/Loko	Paved	28.0	37.0	450	Double Overlay	360	3	
Targrin - Lungi	Paved	9.6	17.8	700	Double Overlay	295	3	
Wellington - W/loo	Paved	2.2	16.2	2312	Double Overlay	324	4	
	Paved	16.2	26.2	1540	Double Overlay	360	3	

RECOMMENDED PRIORITY LIST FOR YEAR 5

ROAD NAME	TYPE	FROM (km)	TO (km)	TRAFFIC VOLUME	IMPROVEMENT POLICY	IMP. COST ($1000)	NEED YEAR	OTHER YR. TO IMPROVE RD.
Hill St. - Hastings	Paved	0	7.5	240	Single Overlay	180	4	
Masiaka - P/Loko	Paved	0	10.0	800	Double Overlay	360	4	
	Paved	19.3	28.0	700	Single Overlay	208	5	
	Paved	37.0	45.0	450	Single Overlay	192	5	
F/town - Penin. Rd.	Paved	13.5	22.2	370	Single Overlay	268	4	
	Paved	22.2	30.2	370	Single Overlay	192	5	

* 2 (10) - This means road will be improved again in the second year with 10 cm. of gravel material.

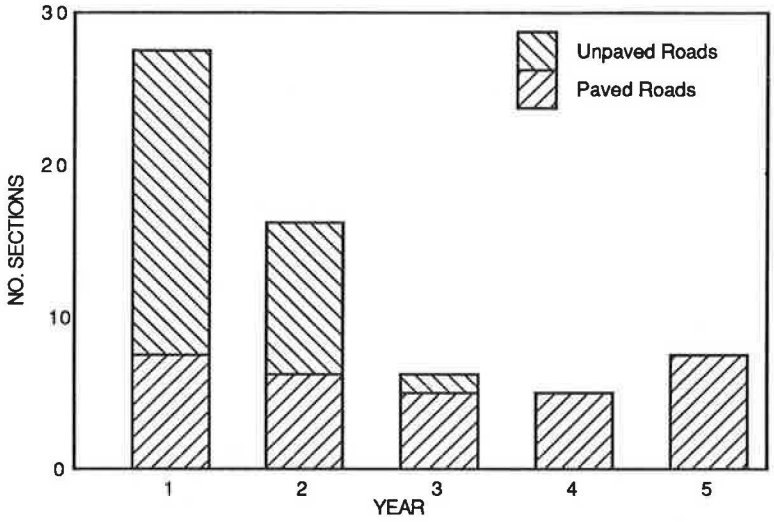

FIGURE 7 Distribution of priority list for a fixed annual budget of $3.25 million.

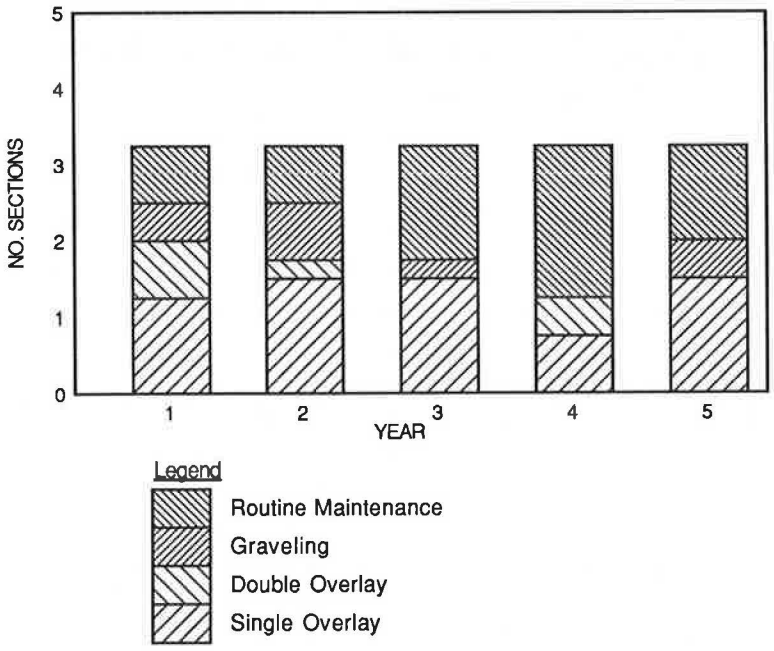

FIGURE 8 A breakdown of improvement costs on the priority list by paved and unpaved roads.

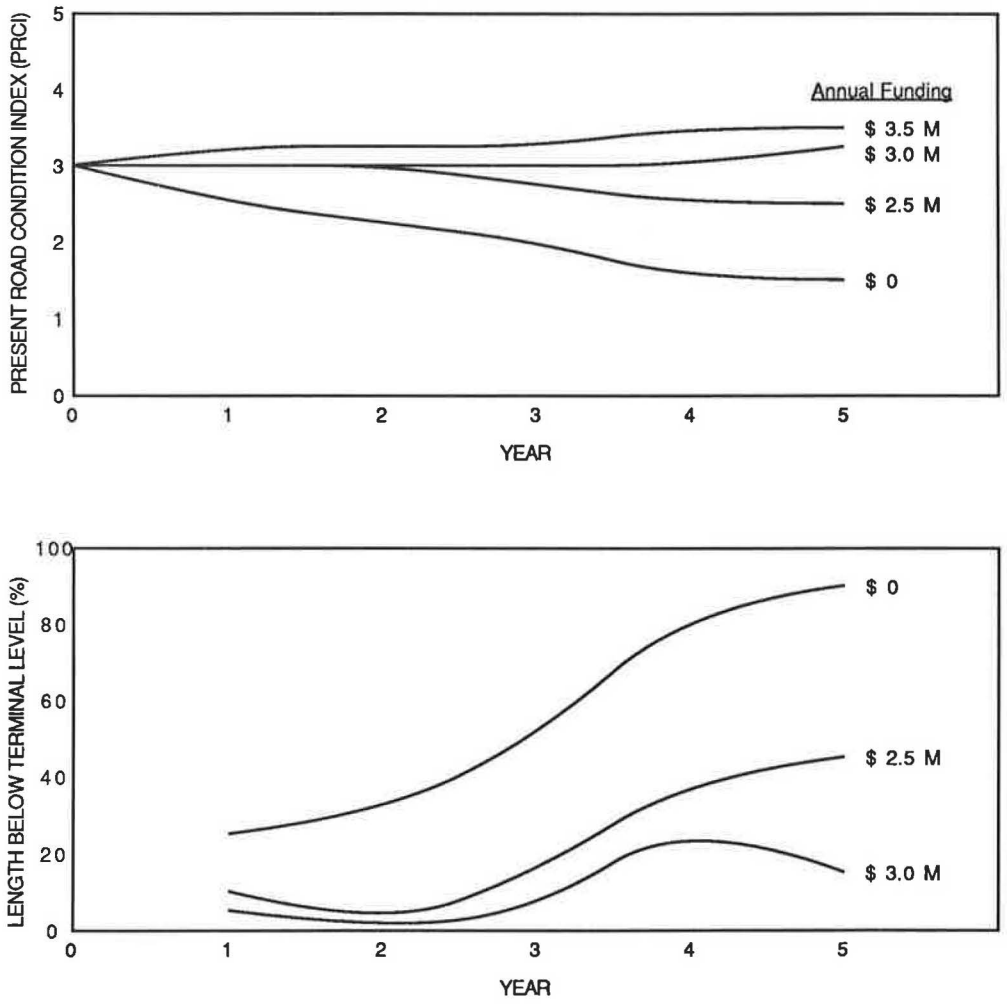

FIGURE 9 The effects of funding decisions on the average condition of the paved road sector.

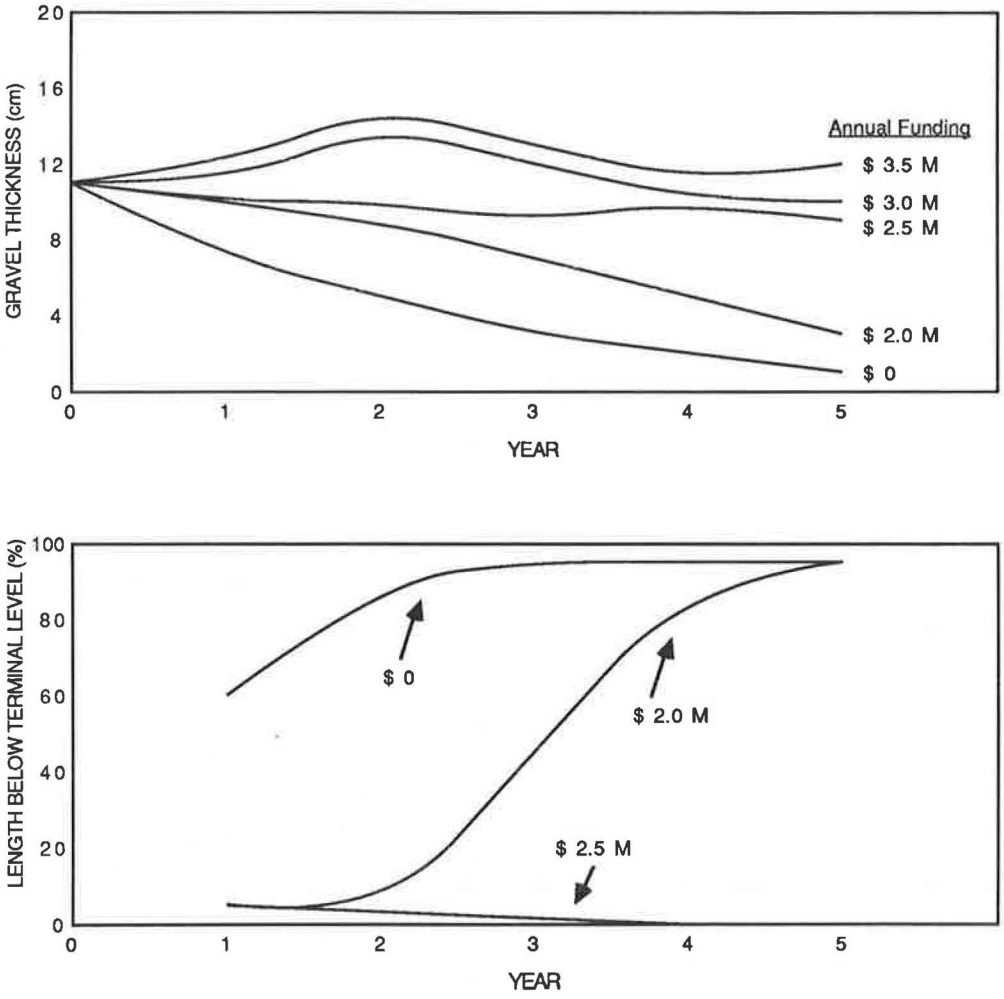

FIGURE 10 The effects of funding decisions on the average condition of the unpaved road sector.

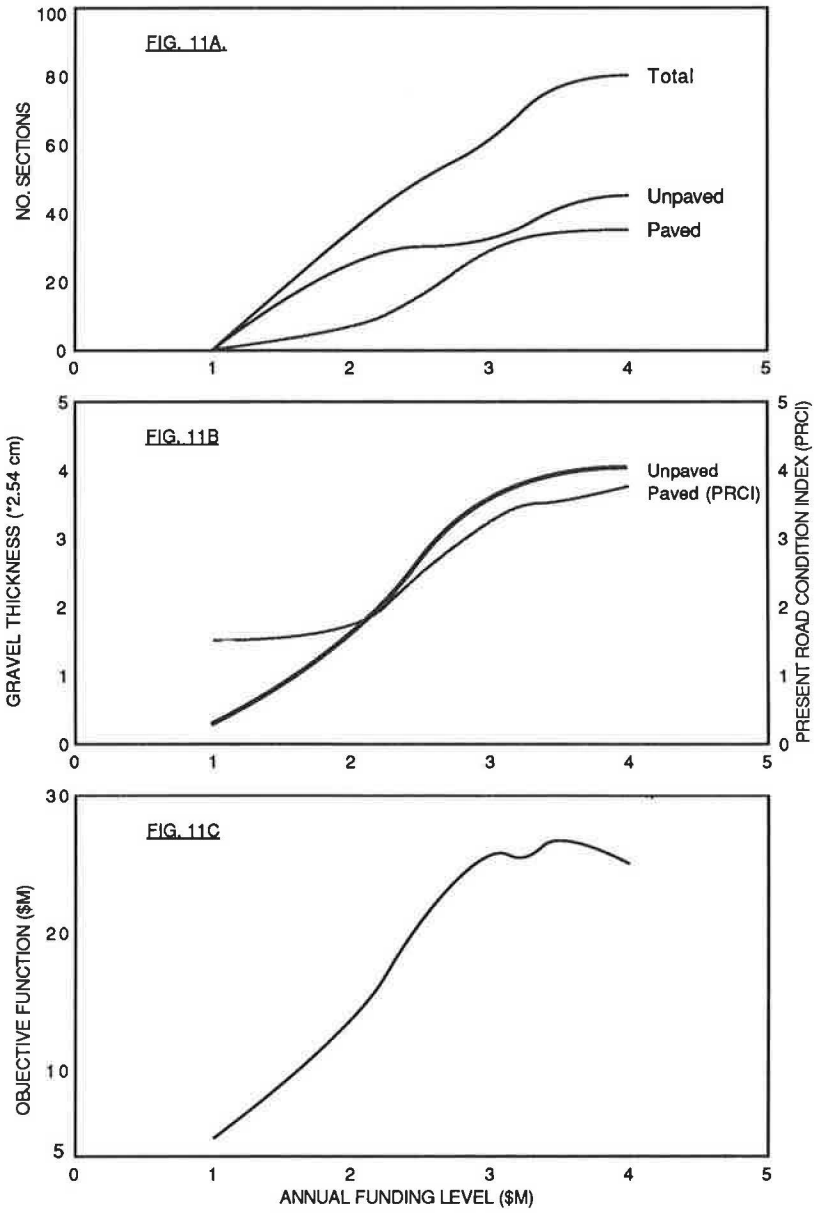

FIGURE 11 The effects of funding decisions on the average condition of the sample road network.

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Economic Viability of Upgrading Low-Volume Roads

H. R. KERALI, M. S. SNAITH, AND R. C. KOOLE

In many countries, particularly in the developing world, a large percentage of the road network is made up of gravel roads. Although such roads are suitable for low traffic volumes, many countries have had rapid increases in traffic levels. This process has resulted in large expenditure on maintenance and rehabilitation of gravel roads as well as high costs of vehicle operation. At high traffic levels, it is therefore desirable to upgrade such roads with bitumen or asphalt pavements to reduce total costs. A computer package incorporating the World Bank Highway Design and Maintenance Standards Model (HDM–III) has been developed to determine the traffic levels at which it becomes economic to upgrade road pavements. Depending on the level of accuracy required, the computer package may be used to give approximate or detailed results. At the simplest operation level, only a few data variables are required to accomplish a quick analysis giving approximate results. A graphical plot of the total costs is displayed on the computer screen showing the crossover point at which upgrading becomes viable.

The purpose of constructing a road is to provide safe and reliable vehicle movement. In many countries, roads have to be constructed along route corridors that pass through areas with soils that are both weak and susceptible to the ingress of water. The construction of an appropriate standard of pavement is intended to provide a smooth riding surface in addition to protecting the underlying soil subgrade from the adverse effects of traffic loading and excessive moisture. The function of a highway engineer is therefore to design cost-effective roads with appropriate geometric and pavement standards that meet the objectives. In order to perform this function, there are a number of different road types lying between engineered gravel roads and heavy-duty bituminous or asphalt pavements that the highway engineer can adopt in the pavement design stage, depending on the economic viability of the proposed road.

The progression from one of the above pavement standards to the next represents a significant improvement in the capacity of a road to withstand traffic loading and the adverse effects of the environment. An increase in construction expenditure would be required, but this is countered by lower rates of pavement deterioration caused by traffic loading and the environment. Each successive improvement in pavement standards may therefore be justified by consequent savings in maintenance expenditure, in the cost of vehicle operation, and in other road user costs. Hence, it is clear that with high traffic, or severe environment, there is a greater likelihood that construction of a stronger pavement would be justified.

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In many developing countries, a large percentage of the road network is unsealed (earth or gravel roads), and despite the generally low traffic volumes on such roads, there is a need to protect them from being washed away, particularly in hilly or mountainous regions with high rainfall. The ingress of water can be prevented by sealing a road surface using a relatively cheap bituminous layer such as surface dressing or other surface treatment. With increased traffic loading, a road requires additional strength to withstand the development of permanent deformation in the subgrade and in other underlying granular layers. In order to reduce the stresses, an extra thickness of pavement is required. This is usually provided by gravel in sealed or surface-dressed roads, but even gravel is becoming increasingly scarce and requires continual replenishment. At a certain combination of gravel cost and traffic, it becomes cheaper to add the required strength to the road in the form of a load-spreading structural bituminous layer.

A simplified method of economic appraisal has been developed for calculating the costs and benefits of the transition from unsealed roads to sealed roads and then further to roads with a structural bituminous load-bearing pavement layer. The transition point is defined in terms of the number of vehicles using a road per day required to justify a higher pavement standard. This is referred to as the break-even traffic level for upgrading from a weak to a stronger type of pavement. For a given set of circumstances, the transition point between pavement standards depends on the costs of road construction, road maintenance traffic composition, and the resulting vehicle operating costs (VOCs). These costs will largely depend on the location of the project together with the effects of topography and climate. Several economic analyses were conducted to determine typical break-even traffic levels in a number of environments. The results are intended for use in chart form to identify situations in which upgrading a road pavement structure may be beneficial.

SHELL INPUT DATA PROGRAM

A user-friendly microcomputer program has been developed to be used as a front-end data preparation tool. The primary objective of the Shell Input Data (SID) program is to prepare data for the World Bank HDM–III model in the form required for the break-even analysis. HDM–III is then used to calculate the transport costs incurred on any two pavement types at different traffic flow levels. The package therefore provides a user-friendly front end that can be used to run HDM–III with the specific purpose of determining the optimum traffic levels for upgrading road pavements under different environments.

TABLE 1 TYPICAL DATA BANDS USED IN SID, LEVEL 2

Parameter		Level 2 Default Values		
		Low	Average	High
Environment:				
Rainfall	(mm/month)	50	100	200
Altitude	(mm)	200	1000	2000
Geometric Characteristics:				
Horizontal Curvature	(deg/km)	50	180	500
Vertical Alignment	(m/km)	5	10	30
Carriageway Width	(m)	2.5	3.5	4.5
Pavement Standard:				
Surface Layer Thickness	(mm)	25	50	100
Roadbase Thickness	(mm)	100	250	500
Subbase Thickness	(mm)	100	250	500
Subgrade CBR	(%)	5	15	30

The SID program provides users with three modes of operation (Levels 2, 1, and 0), corresponding to the simplicity required in data processing. In Level 2 operation, only a few parameters required to run HDM–III are requested; other input parameters are assigned default values. Data entry using Level 2 is intended to give approximate results from which a decision may be made for further investigation. In this mode, only a few data cards can be edited to specify parameters such as the unit costs of construction, maintenance, vehicle operation, and traffic characteristics. Other physical characteristics of the road to be analyzed can be selected from low, average, or high values. Table 1 presents examples of default values suggested by the SID program in Level 2 operation. The default values are intended to give the orders of magnitude believed to be typical for the area of analysis and can be changed by the user. However, Level 1 operation requests all parameters necessary to run HDM–III and is designed to provide more accurate results. It is intended to be used by experienced HDM–III users, who are familiar with all input requirements. Levels 1 and 2 are tailored to provide data to HDM–III for the comparison of any two types of pavement structure to obtain the break-even traffic level. Level 0 of the data input program can be used to specify input parameters to HDM–III for any type of economic analysis (e.g., the appraisal of maintenance alternatives).

ECONOMIC ANALYSIS OF BREAK-EVEN TRAFFIC LEVELS

The economic analysis involved in deriving the break-even traffic level is a straightforward comparison of transport costs calculated for any two road types. For example, to determine whether a gravel road or a sealed road is more suitable under a given set of circumstances, HDM–III is used to calculate the annual transport cost matrices for the two alternatives. For example, if the present value of costs for a gravel road alternative exceeds that of a sealed road, it can be concluded that, under the given set of circumstances, a sealed road has lower total transport costs and therefore its construction would be economically viable.

In this example, the set of circumstances refers to a road with known climate, topography, and traffic flow characteristics. The method of appraisal could therefore be used to answer the question, Is it cheaper to operate a gravel or a sealed road under the present traffic loading pattern? In order to answer the next question—At what traffic flow level does it become cheaper to operate a sealed instead of a gravel road?—it is necessary to calculate the total transport costs for both road types for a range of traffic flow levels and to determine the average daily traffic (ADT) at which the transport costs for the two pavement types are equal.

Weak pavements deteriorate faster than strong pavements under traffic loading and other environmental effects. At low traffic levels, the rate of pavement deterioration is low, and therefore transport costs are likely to be low. Consequently, by comparing the transport costs calculated for a range of traffic loading on two pavement types, it is possible to interpolate and obtain the ADT at which transport costs are equal for the two pavement types.

The economic analysis required to determine the break-even traffic is conducted by using HDM–III to calculate transport costs for any two pavement types under a range of traffic flow levels. Table 2 presents the default traffic flow levels used in HDM–III to compare transport costs calculated for gravel, sealed, and premix roads. These values represent initial ADT with a specified traffic composition, that is, observed traffic flow in the first year of analysis (the base year). The traffic levels were chosen to include the upper and lower limits for the break-even ADT likely to be found in most countries. When these are used in a particular country, the unit costs of construction, maintenance, and vehicle operation are required together with the maintenance standards commonly applied. These unit costs, maintenance standards, and traffic mix affect the break-even ADT level. For countries in which costs of vehicle operation are high, a low break-even ADT can be

TABLE 2 TRAFFIC FLOW LEVELS USED IN HDM-III COMPARISONS

Alternative	Gravel to Sealed	Sealed to Premix	Gravel to Premix
ALT 1	20	100	100
ALT 2	50	200	200
ALT 3	75	400	400
ALT 4	100	700	700
ALT 5	200	1000	1000
ALT 6	400	2000	2000
ALT 7	700	5000	5000
ALT 8	1000	10000	10000

expected. Similarly, if the maintenance costs for the weaker pavement are high (e.g., cost of regraveling), a low break-even traffic can also be expected.

Typical Results Using Data From Cyprus

Trial runs of the HDM-III program were conducted using data from Cyprus with the objective of determining typical break-even traffic flow levels for upgrading from gravel to sealed and from sealed to premix pavements. For these analyses, the gravel road was assumed to have 150 mm of gravel surfacing; the sealed road, 25 mm of surface dressing on 150 mm granular road base; and the premix road, 40 mm of premix surfacing layer on top of 300 mm of selected granular road base material. The results from the trial runs produced for a comparison of a sealed road against a gravel road by HDM-III are summarized in Table 3. The break-even traffic level is given by the ADT level for which the net present value (NPV) is zero (i.e., when the transport costs of the two pavements being compared are equal). It may be observed from the table that the optimum or break-even traffic level required to justify the construction of a surface-dressed road varies with the discount rate used in the analyses. If the economic costs are not discounted, the break-even traffic flow is just over 50 vehicles per day (vpd).

Presentation of Results

There are a number of ways in which the results given in Table 2 can be presented to illustrate the break-even traffic level. Three of these are of particular interest when presented in graphical form:

- Total transport costs versus initial ADT flows,
- NPV versus initial ADT flows, and
- Internal rate of return (IRR) versus initial ADT flows.

Figure 1 shows the first of these with the total transport costs for the gravel and sealed roads on the vertical axis. This graph is similar to the graph produced on the computer screen. The point at which the two curves intersect marks the break-even ADT value. Figure 1 indicates that the break-even traffic level is just under 200 vpd for a 2 percent annual traffic growth rate. Figure 1 can also be used to estimate the size of benefits to be derived by upgrading a road. If the daily traffic flow is known, the benefits derived from upgrading are given by the difference in total transport costs.

Figure 2 shows the second method of presentation with the NPVs at four discount rates plotted on the vertical axis. The break-even traffic level for each of the discount rates is given by the points at which the curves cross the horizontal x-axis when the NPV is zero. The 10 percent discount line crosses the x-axis at a traffic flow of just under 100 vpd.

TABLE 3 COMPARISON OF A SEALED ROAD WITH A GRAVEL ROAD FOR 2 PERCENT TRAFFIC GROWTH RATE

Initial Traffic	Net Present Value at Discount Rate of 0%	5%	10%	15%	I.R.R (%)
20	-0.215	-0.217	-0.218	-0.219	-20.7
50	-0.134	-0.163	-0.179	-0.189	-8.6
75	-0.064	-0.119	-0.149	-0.166	-3.3
100	0.024	-0.066	-0.114	-0.141	1.1
200	0.296	0.107	0.007	-0.051	10.5
400	1.362	0.754	0.435	0.254	32.6
700	3.527	2.081	1.322	0.891	71.4
1000	5.817	3.502	2.281	1.584	109.2

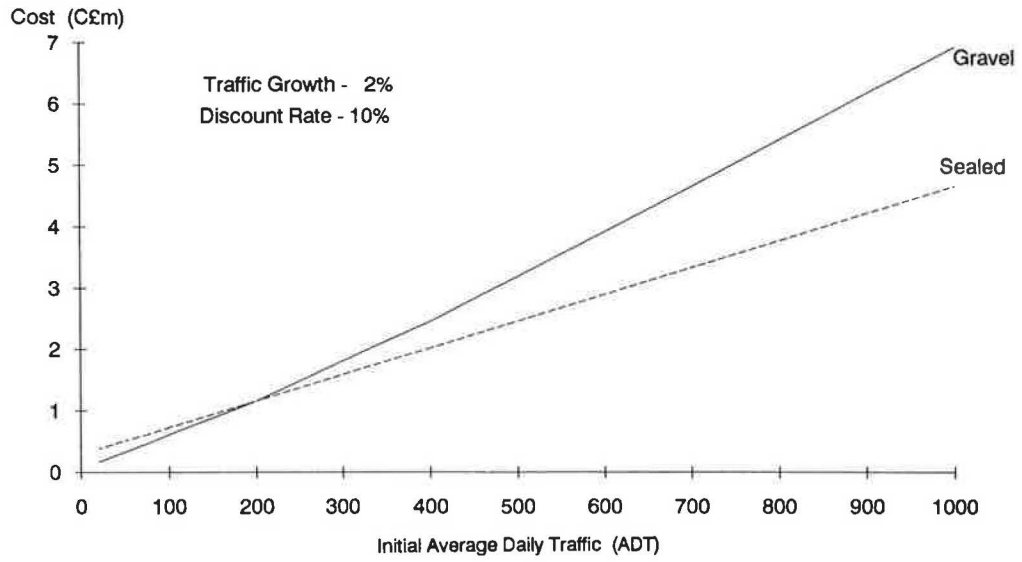

FIGURE 1 Relationship between total transport cost and traffic level.

Table 3 presents IRR values calculated for each of the initial ADT flows. The IRR values represent the discount rates at which the corresponding ADT would justify upgrading from a gravel to a sealed road. Figure 3 shows the relationship between IRR and traffic flow for Cyprus conditions. The figure represents all ADT levels required to justify upgrading a gravel road to a surface-dressed road in Cyprus for any given traffic growth and discount rate. For example, at a discount rate of 10 percent, the break-even traffic flow for surface dressing is just under 200 vpd for a 2 percent growth in traffic.

Factors Affecting the Break-Even Traffic Level

The break-even traffic level discussed in the previous section depends on a number of factors, notably the unit costs of construction and maintenance, traffic growth rates, discount rates, and the traffic composition that has a direct effect on the VOC component. The unit costs of construction and maintenance in most cases are determined before the analysis is conducted and are then assumed to remain constant. The break-even traffic level therefore depends largely on traffic growth and the discount rates used in the economic analyses.

If the VOC component is included in the analyses, the size of the total saving in VOC will depend on the traffic growth rate. If high traffic growth rates are used, the traffic loading during the analysis period will be high, resulting in a faster deterioration of the weaker pavement and giving higher savings in VOC; consequently, upgrading to a stronger pavement will be justified at lower traffic flow levels. Figure 3 shows the effects of these factors on the break-even traffic levels.

EFFECT OF ENVIRONMENT ON BREAK-EVEN TRAFFIC LEVELS

Economic analyses were conducted using HDM–III to determine the break-even traffic for a range of environments using

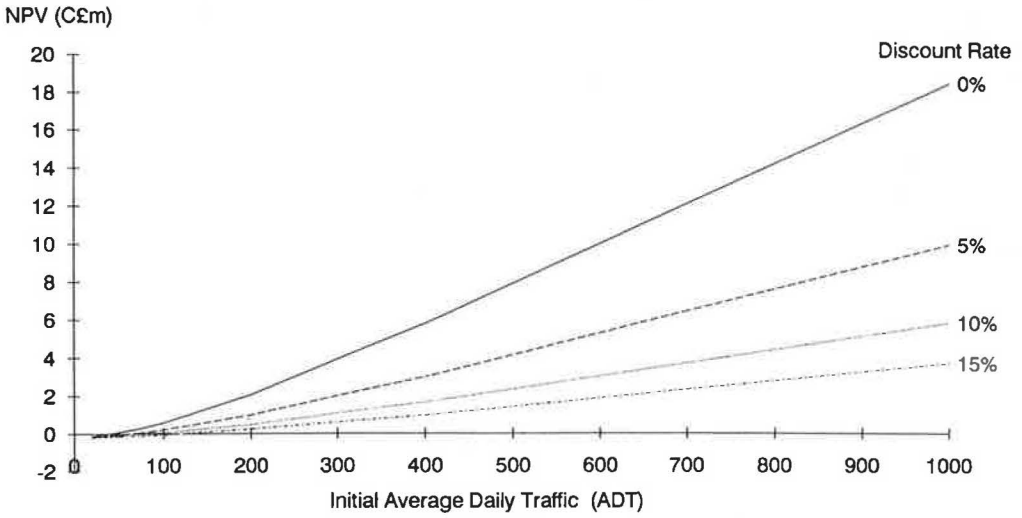

FIGURE 2 Relationship between NPV and traffic level.

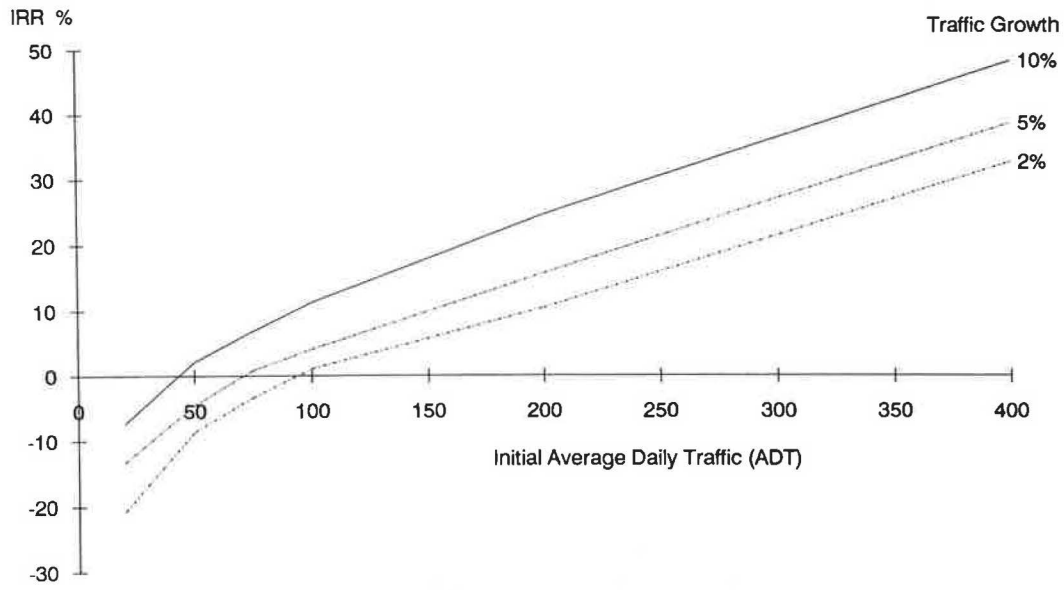

FIGURE 3 Relationship between IRR and traffic level.

data from Cyprus. The objective for this procedure was to quantify the effects of various environmental factors on the break-even traffic levels. The factors included in the analysis were rainfall, topography, and subgrade soil strength in terms of the California bearing ratio (CBR). In the course of the analysis, it was found that some of the environmental factors had little or no effect on the break-even traffic results (e.g., rainfall on sealed or premix roads).

The Shell pavement design manual recommends pavement layer thicknesses based on four factors:

- Traffic loading in terms of equivalent standard axles,
- Subgrade strength modulus,
- Prevailing weather conditions in terms of the weighted mean temperature, and
- Bitumen and mix properties.

The effect of traffic loading and subgrade strength on the design thickness for pavements used in the break-even analysis is discussed in the next section. The effect of temperature on the performance of bituminous pavements is not explicitly modeled within HDM–III. However, temperature variations in different climatic regions can be taken into account by calibrating the performance equations built into the model, such as rut depth progression and other defect progressions. The effects of the environment are also taken into account in terms of the rainfall and altitude. In HDM–III, rainfall is assumed to affect the performance of unpaved roads, particularly the rate of gravel loss.

Pavement Design Standards

Traffic loading is used in the break-even analysis as the independent variable that governs the size of benefits derived from upgrading a road from one pavement type to another. The normal practice is to design pavement thicknesses according to predicted cumulative traffic loading and prevailing subgrade strength conditions over the design period. The break-even analysis is essentially conducted by calculating the total transport costs incurred on a road using a range of traffic flow levels. Ideally, each traffic range used in the analysis should be assigned a unique pavement design (and hence cost of construction). However, to maintain the simplicity of the calculation of the break-even traffic level, the same pavement structure is used in all analyses. This pavement structure represents the minimum possible design thickness. For example, the minimum pavement structure for premix roads is assumed to include a 50-mm asphalt concrete (or equivalent) surfacing and 300 mm of granular road base. The minimum sealed road structure is a surface dressing of 25 mm with a granular base of 300 mm. For unpaved roads, the minimum gravel thickness commonly used is 150 mm.

This assumption is compensated for in the analyses by using a condition-responsive maintenance policy within HDM–III. The effect is to apply frequent maintenance and rehabilitation on underdesigned pavements when the pavement attains critical condition. For example, a pavement composed of 50 mm of asphalt concrete and 300 mm of granular road base is used for ADT levels from 100 to 5,000 but is expected to receive more overlays during the analysis period under the heavier traffic load.

Effect of Rainfall and Topography

Only rainfall in combination with high vertical alignment had noticeable effects on the break-even traffic level. The analyses were therefore performed for a combination of low rainfall in a flat area and high rainfall in a mountainous area. The results obtained for intermediate combinations of rainfall with topography indicated deviations in the break-even traffic level of less than 20 vpd.

The break-even traffic levels for upgrading roads from gravel to sealed pavements range from 50 to 150 vpd depending on

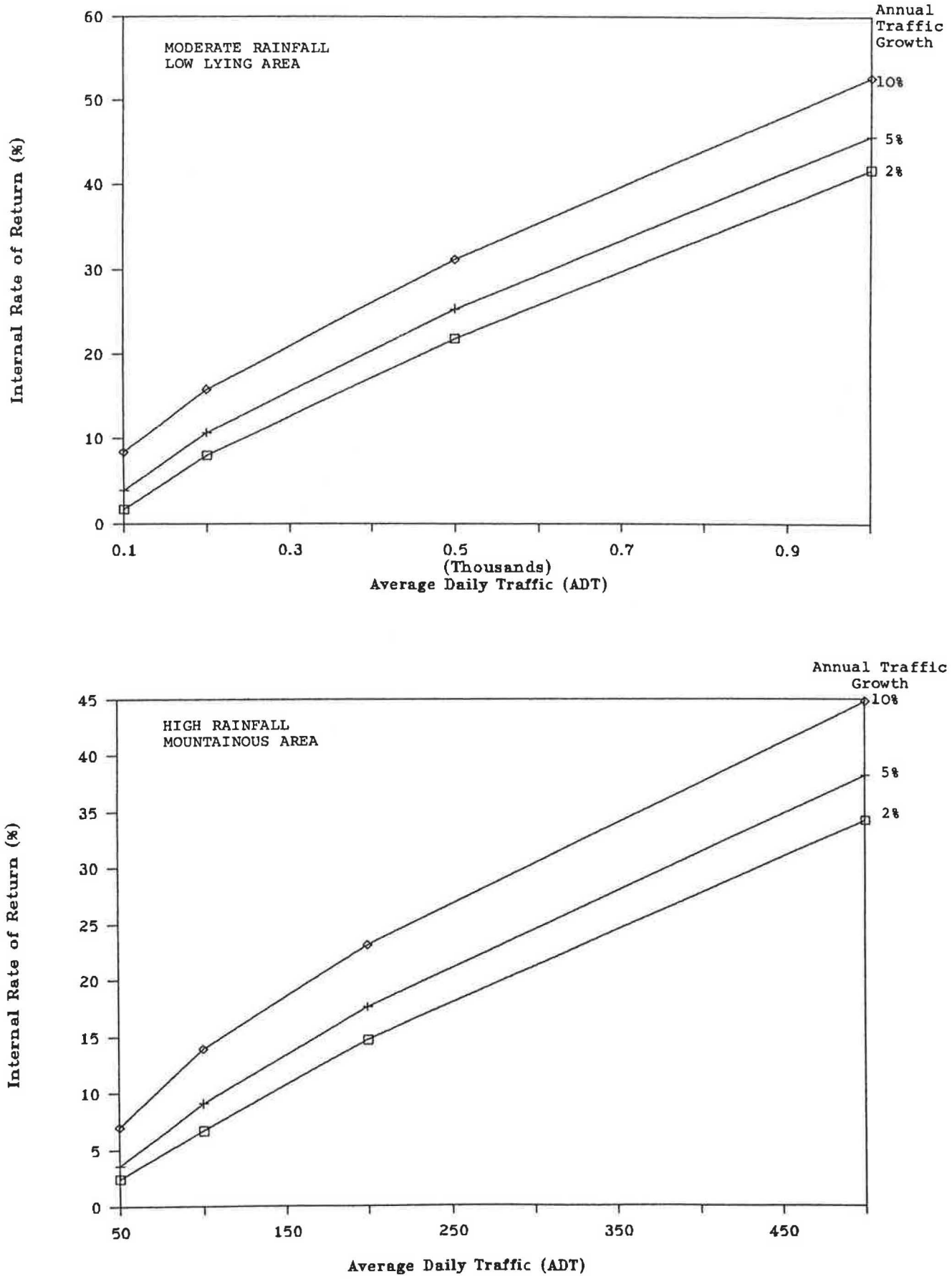

FIGURE 4 Effect of rainfall and topography on break-even traffic level: (*top*) upgrading gravel to premix road and (*bottom*) upgrading gravel to sealed road.

the traffic growth rate and the discount rate used in the analysis. The combined effect of rainfall and severe topography is to lower the break-even traffic level by between 20 to 50 vpd, again depending on the traffic growth rate and the discount rate. Figure 4 shows the results of break-even analyses conducted for a low-lying area with moderate rainfall and for a mountainous area with high rainfall.

The comparison of sealed-road performance against that of premix roads gave some unexpected results, because the break-even traffic level from sealed to premix was between 500 to 750 vpd. This comparison refers to premix roads with the minimum pavement structure (i.e., 50 mm asphalt concrete on 300 mm granular road base). This result is in contrast with ADTs of over 1,500 vpd, commonly quoted as the minimum for premix standard pavements.

The break-even traffic level for upgrading from a gravel to a premix road is surprisingly low, ranging from 100 to 250 vpd. This result indicates a rapid increase in transport costs once the break-even traffic level for paving a road is exceeded. It therefore suggests that sealing a road at an early stage will yield high benefits.

The preceding results point to a high degree of dependence on the unit costs of construction used for gravel roads, sealed roads, and premix roads together with the unit costs for maintenance activities. Gravel roads, though least expensive to construct, have a high VOC component because of high surface roughness values. The difference in roughness progression on sealed and premix roads, however, is similarly high, resulting in a lower break-even traffic level than is commonly expected.

CONCLUSIONS

A computer package incorporating the World Bank's HDM–III model can be used to determine the traffic levels at which the transition from one pavement standard to the next becomes economic. A number of analyses were conducted using the package to study the effects of the climate, topography, and subgrade soil strength on the break-even traffic levels. The results of the analyses indicate that the break-even traffic level for upgrading roads from gravel to sealed is surprisingly low, implying a rapid increase in transport costs once the break-even traffic level for paving a road is exceeded. This confirms the view that sealing a road at an early stage is advisable. The combined effects of heavy rainfall and severe topography result in a lower break-even traffic level than might be expected. The comparisons of sealed-road performance and that of premix roads also gave some unexpected results, because the break-even traffic level from sealed to premix occurs at much lower ADT flow levels than is commonly expected.

ACKNOWLEDGMENTS

The work described in this paper was sponsored by Shell International Petroleum Company (SIPC). The Highways and Geotechnics Research Group of the University of Birmingham is greatly indebted to SIPC for providing the resources required in the research study.

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Maintenance of Minor Roads Using the Lengthman Contractor System

T. E. JONES AND R. C. PETTS

Current development and research activities on the Minor Roads Program in Kenya are described. The relatively rapid deterioration of unpaved roads compared to paved roads demands that maintenance be programmed rather than respond to changes in surface condition. Road maintenance in developing countries is normally organized on a district or regional basis using ministry teams or contractors. However, the use of lengthmen to undertake routine and possibly recurrent maintenance activities may result in more rapid and cost-effective correction of deficiencies and an improved level of service in some circumstances. The Minor Roads Program is a labor-based improvement and maintenance program for low- and medium-trafficked roads in 28 districts of Kenya with a high agricultural potential. The program commenced in 1986 and is projected to improve 4 500 km of roads to gravel all-weather standard over a 5-year period and to establish maintenance systems. The program will also maintain some 8 000 km of roads constructed under the previous labor-based Rural Access Roads Program. People living alongside the roads are utilized on a casual basis to carry out all improvements and maintenance supported by simple agricultural equipment for certain operations, particularly on the higher-trafficked roads. Routine maintenance on the roads is carried out using a lengthman system. An ex-construction worker is appointed to each section, typically 1.5 to 2.0 km in length. He is provided with the necessary hand tools and is paid for working 3 days per week on the road. This allows him time to continue to work on his land on other days. The principal aim was to establish productivity standards for a range of routine and recurrent maintenance activities carried out by lengthmen using mostly hand tools. The current research will evaluate the influence of geometry, climate, traffic, and materials on the amount of maintenance required and define optimum lengths of road relative to labor inputs for these variables.

A network of more than 8 000 km of rural access and minor roads has been constructed and maintained in Kenya using labor-based methods. Routine maintenance of these roads is carried out using individual lengthmen contractors. Current research work is designed to establish maintenance needs and productivity standards. The results of the research will permit improved planning, implementation, and monitoring of labor-based road maintenance.

The use of individual contractors or lengthmen to undertake routine maintenance over specified lengths of road has been a recognized management technique for many years. However, the effectiveness of the technique over a long period has seldom been monitored in detail.

The Kenyan Rural Access Road Program (RARP) was started in 1974. Its primary objective was to construct farm-to-market access roads using a labor-based method in districts with high agricultural potential. By the end of 1986, approximately 8 000 km of rural access roads had been completed in 26 districts with the majority of them graveled and maintained by individual lengthmen contractors.

Following the success of the RARP in constructing roads at low cost and with high utilization of local resources, the government of Kenya decided to apply the same labor-based methods to the improvement and maintenance of selected D and E roads (Table 1), which are categorized as minor roads (*1*). During the period 1986 to 1992, the Minor Roads Program (MRP) is projected to improve 4 500 km of minor roads and to include them in Kenya's overall maintenance strategy.

The research is concerned with the initial establishment of maintenance productivity standards in the MRP and the determination of the routine maintenance needs of rural access and minor roads (RA/MRs).

For the MRP, routine maintenance is defined as all work required within the road margin capable of being carried out by an individual lengthman contractor. Periodic maintenance of the roads includes regraveling and repairs to structures and this is carried out by special units or contracting companies. Occasionally, urgent work is carried out under various arrangements when the scope is beyond the capabilities of the lengthman contractor.

DEVELOPMENT OF THE MAINTENANCE SYSTEM

Various methods were originally considered for the routine maintenance of rural access roads. For the chosen lengthman system, an ex-construction worker was appointed on a contract basis to each section of road, typically 1.5 to 2.0 km in length. He was provided with hand tools and supervised once a month by an overseer to monitor the condition of the road and to authorize payments for satisfactory work. The payment was based on the contractor carrying out 12 days of work per month on days of his choice. The contractor could be replaced if he consistently performed badly.

The contractor lived adjacent to the road and would therefore not require government accommodation or transport, which consumed considerable resources in a traditional equipment-based maintenance system.

A principal attraction of the system is the comparatively low level of equipment required and consequently lessened support problems. This advantage is coupled with a low foreign exchange component, which in 1981 was estimated to be only 10 percent (*1*). This amount compares with a typical

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TABLE 1 KILOMETERS OF KENYA ROAD NETWORK UNDER THE RESPONSIBILITY OF THE MINISTRY OF PUBLIC WORKS

CATEGORY	BITUMEN	EARTH/GRAVEL	TOTAL
A. INTERNATIONAL TRUNK ROADS (1)	2,608	971	3,579
B. NATIONAL TRUNK ROADS (1)	1,308	1,443	2,751
C. PRIMARY ROADS (1)	2,293	5,476	7,769
D. SECONDARY ROADS (1)	1,041	10,074	11,115
E. MINOR ROADS (1)	512	25,754	26,266
SPECIAL PURPOSE ROADS (1)	166	2,931	3,097
RURAL ACCESS ROADS (2)	15	7,962	7,977
TOTAL	7,943	54,611	62,554

NOTE (1): Network at September 1990.
(2): Network at June 1990.

foreign exchange component for equipment-based routine maintenance systems of 50 percent (*2*).

The lengthman system also creates productive paid employment in rural areas where there are few opportunities for such work. The contractor is able to live at home with his family and the part-time terms give him the opportunity to work on his own land as well.

Approximately 80 percent of the direct costs of the system was estimated to be paid directly to the contractors in 1982. The system also enables maintenance to be achieved throughout the year on each section of road. The responsibility for the maintenance of each road section lies completely with one person who requires minimal logistic support.

Unfortunately, the establishment of the maintenance system did not attract the same amount of research and development effort as the construction aspects of the program. It was erroneously assumed that the local administration and people would bring pressure to bear on the contractors to maintain the roads to a good standard. The contractor's appreciation of the maintenance requirements was taken for granted. The need for training and supervision was underestimated and mechanical problems associated with the supervision vehicles had an adverse effect.

It is now realized that the development of effective maintenance systems requires as much, if not more, effort than construction or road improvement systems. In recognition of these problems, a study of maintenance of rural access roads was commissioned in early 1985 (*3*). The report showed the lengthman system to be quite effective. However, it indicated a significant potential for improvement and the need for better direction and control of the contractors. Extensive discussion of the maintenance issues has enabled the principal problems to be identified and appropriate strategies developed for tackling them.

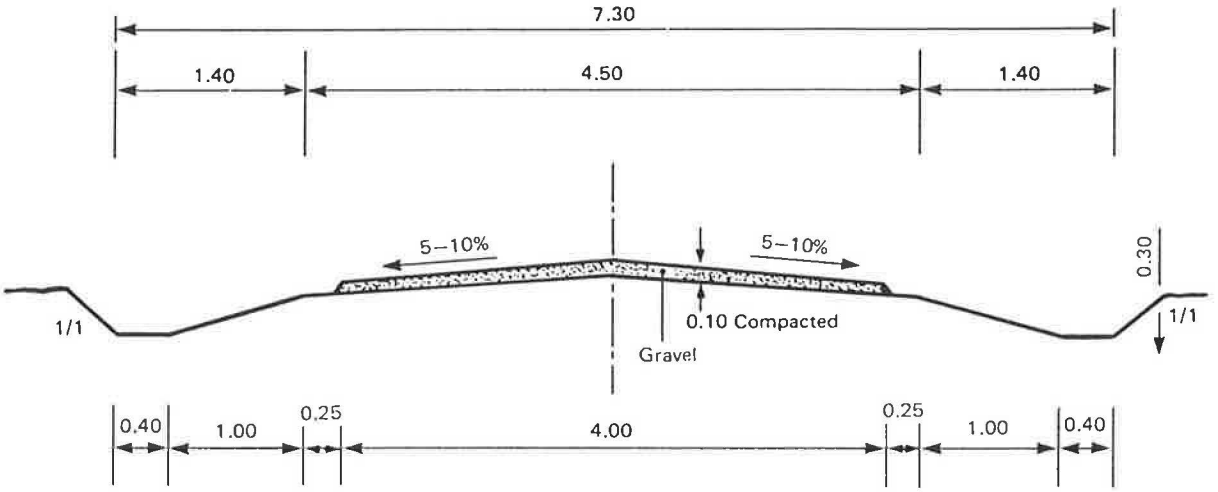
In particular, there was a need to more accurately determine the maintenance requirements of RA/MRs under various conditions of rainfall, alignment, pavement and soil type, and traffic. Rural access roads have a 4-m-wide graveled running surface, whereas the minor road standard is 5.4 m (see Figures 1 and 2).

Methods of determining required maintenance resources and their deployment, direction, and control needed to be developed. Arrangements for dealing with urgent works such as washouts and culvert breakages should be formalized. The methods of identifying spot regraveling and full regraveling required to be developed and the various options for carrying out this work, e.g., by animal-drawn haulage and casual labor or small-scale contractors, needed to be investigated. There was an urgent requirement to ease the supervision burden of the maintenance overseers because of the minimal time that they could allocate to each contractor and the mechanical problems that will always exist to a degree, even with the low equipment component of the system. The scope for the use of headmen, responsible for a small number of contractors, needed to be developed, as well as the methods of training, directing, and monitoring them. On the technical side, there was the problem of maintaining a satisfactory longitudinal profile, especially for the wider, more heavily trafficked minor roads. There was also the question of safety for lengthmen working on the carriageway of the more heavily trafficked roads [>50 vehicles per day (vpd)]. Consideration had to be given to the use of simple tractor-drawn mechanical graders or drags for maintaining the running surface in these circumstances, with pothole-patching support and all off-carriageway work by the lengthmen.

The lengthman concept has been adopted for the maintenance of RA/MRs under the new program. However, a number of major improvements to the system have been initiated or planned. Studies of particular maintenance aspects are being carried out as part of this process.

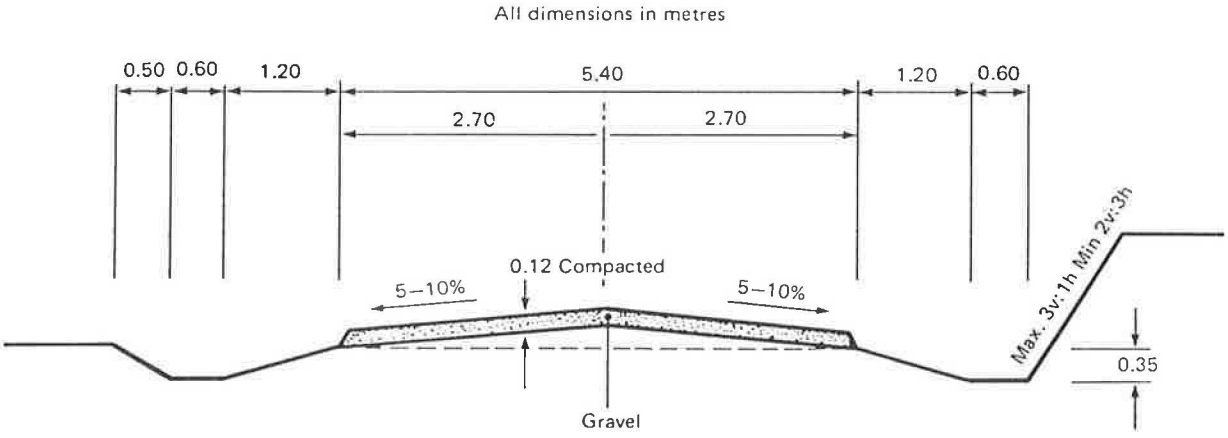
The main areas of improvement to the maintenance system are as follows:

1. The setting of fair contract lengths for the lengthmen including objective consideration of the factors influencing the amount of maintenance required;
2. The supervision of lengthmen activities and concentration on one routine maintenance activity at a time;
3. Arranging for the lengthmen to work progressively from one end of a section to the other on a specified activity, easing supervision and control;
4. The establishment of realistic task rates for routine maintenance activities;



Notes: Camber profile depends on whether constructed from arisings of one ditch or two.
Other sections apply for Black Cotton Soil, severe terrain or embankment situations

FIGURE 1 Rural access road, standard cross section.



Notes: Camber profile depends on whether constructed from arisings of one ditch or two.
Other sections apply for Black Cotton Soil, severe terrain or embankment situations

FIGURE 2 Minor road, standard cross section.

5. The setting of priorities for routine maintenance activities according to season;
6. The establishment of reference stations on each lengthman's section for control and reporting purposes;
7. The introduction of both working and nonworking headmen in appropriate situations to control work between overseers' visits;
8. The introduction of control aids for headmen;
9. The introduction of an objective assessment of routine maintenance performance incorporated in an annual inspection system that will also allow periodic maintenance work to be identified and monitored;
10. A training program including formal and on-the-job training, and demonstration sites in each district; and
11. Allocation of the highest priority to the provision, servicing, and repair of routine maintenance overseers' motorcycles.

Items 1 and 4 are the subject of the current study. Other initiatives are being taken to bring about the other improvements. Phase I of the study has already established routine maintenance productivity standards (task rates). Phase II of the study will monitor the maintenance requirements of RA/MRs under the range of principal influential factors experienced; namely rainfall, gradient, traffic, and surface characteristics. Figure 3 shows the flow diagram for the study. Figure 4 shows the proposed district structure for the established maintenance organization.

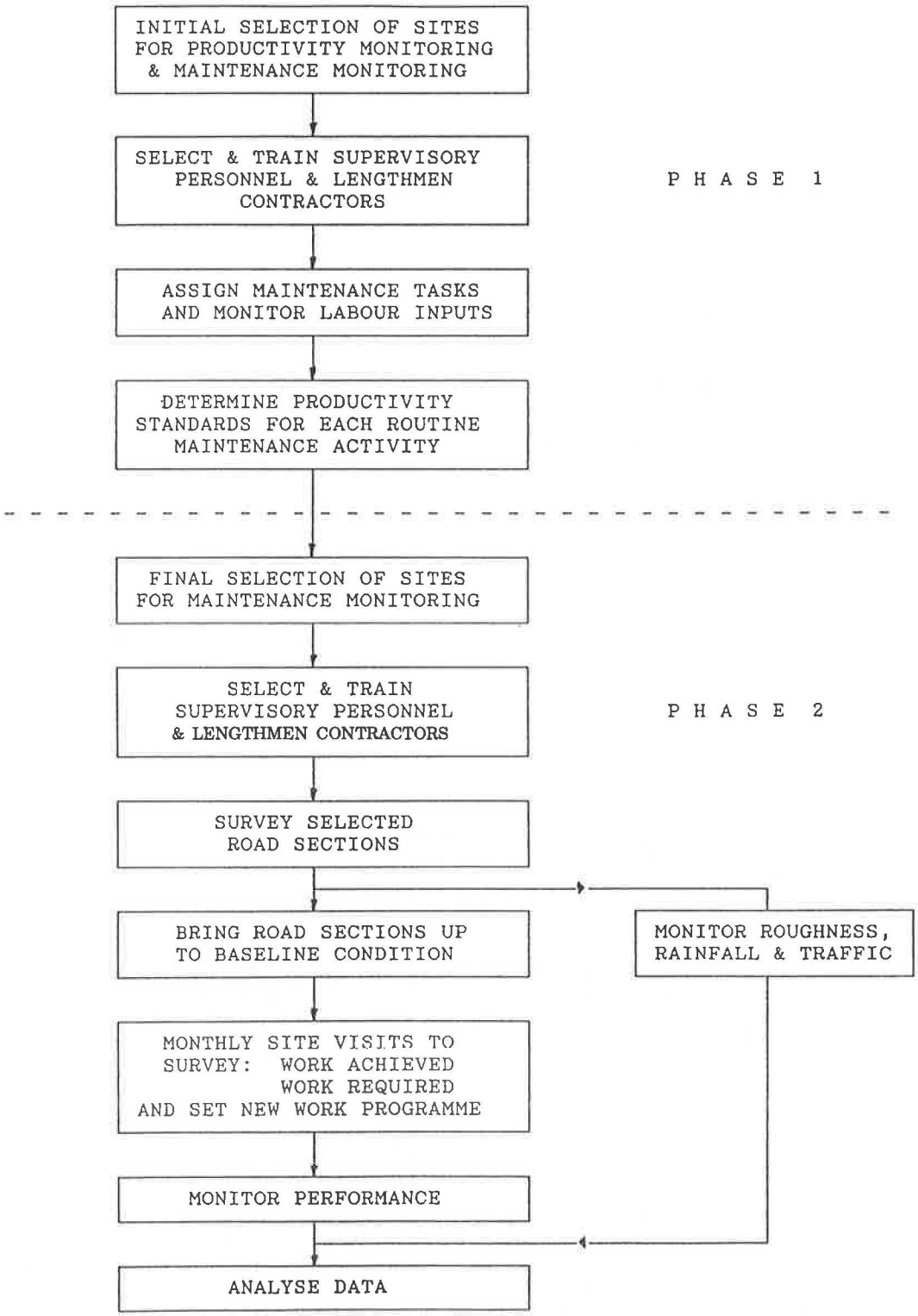
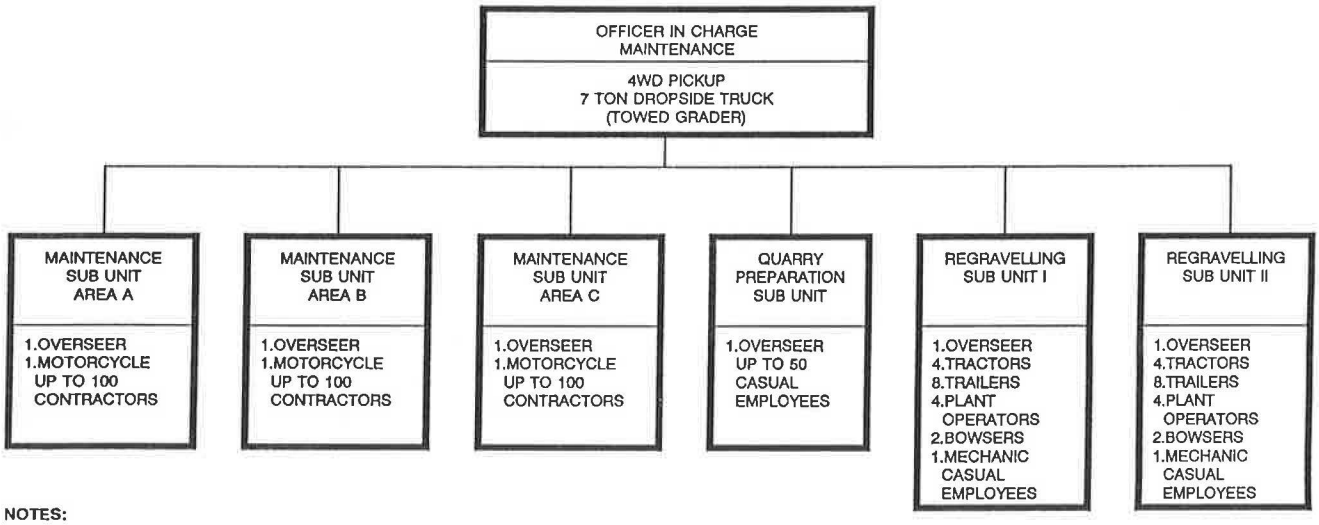


FIGURE 3 Flow diagram of the research program.



NOTES:

1. Maintenance sub units as required.
2. Second regravelling sub unit only if required.
3. Regravelling sub units to also replenish contractor stockpiles.
4. Regravelling resources calculated on 7500m³ of regravelling/stockpiling per sub unit year.
5. Bowsers: single axle towed, 1 fuel and 1 water per sub unit.

FIGURE 4 Proposed MR/RAR program district maintenance unit.

PHASE I: ESTABLISHMENT OF MAINTENANCE PRODUCTIVITY STANDARDS

Objective

Phase I of the study concentrated on the following objective: to establish productivity standards for a range of routine maintenance activities utilizing hand tools. The following activities were identified as the principal routine maintenance operations for RA/MRs for which productivity levels were established in the study:

Activity Code	*Description*
R1	Inspection and removal of obstructions
R2	Clean culverts and inlets and outlets
R3	Repair culvert headwalls
R4	Clean miter drains
R5	Clean side drains
R6	Repair scour checks and side drain erosion
R7	Repair erosion on shoulders
R8	Fill potholes in carriageway
R9	Grub edge and reshape carriageway
R10	Cut grass in the side drains
R11	Clear bush

Some of the operations are shown in Figures 5–12.

Methodology

Phase I of the study was conducted in Kisii and South Nyanza districts between October 1988 and March 1989, a period that conveniently overlapped wet and dry seasons in both districts. Kisii is a high-rainfall district with generally cohesive "red coffee" soils at approximately 1800-m elevation and has an annual rainfall of between 1 500 and 2 000 mm. South Nyanza is a drier district with predominantly sandy and "black cotton" soils at approximately 1 100 m elevation, annual rainfall there

FIGURE 5 Lengthmen working collectively.

FIGURE 6 Grubbing edge.

FIGURE 7 Excavating gravel for patching.

FIGURE 10 Cleaning culvert inlet.

FIGURE 8 Reshaping carriageway.

FIGURE 11 Cleaning miter drain.

FIGURE 9 Cleaning culvert outlet.

FIGURE 12 Towed grader under trial.

being between 1 000 and 1 500 mm. Program districts are shown in Figure 13. The two districts are considered to be typical of many MRP districts so that the productivity data derived may be applicable across the program.

Three overseers were seconded to the study team, each supervising up to 14 maintenance contractors with the assistance of two nonworking headmen. This ratio contrasts with the supervision ratio planned for the MRP, which would typically be 1 overseer to 8 headmen to 80 lengthmen. In some areas, working headmen have their own maintenance section as well as supervision responsibilities over other lengthmen.

The contractors were generally working on their own separate sections before the study. However, for ease of supervision, during the study they were brought together in small groups of seven under each nonworking headman. Before the commencement of data collection, the contractors and headmen were given approximately 2 weeks of training by the overseers. Various levels of difficulty were defined for each

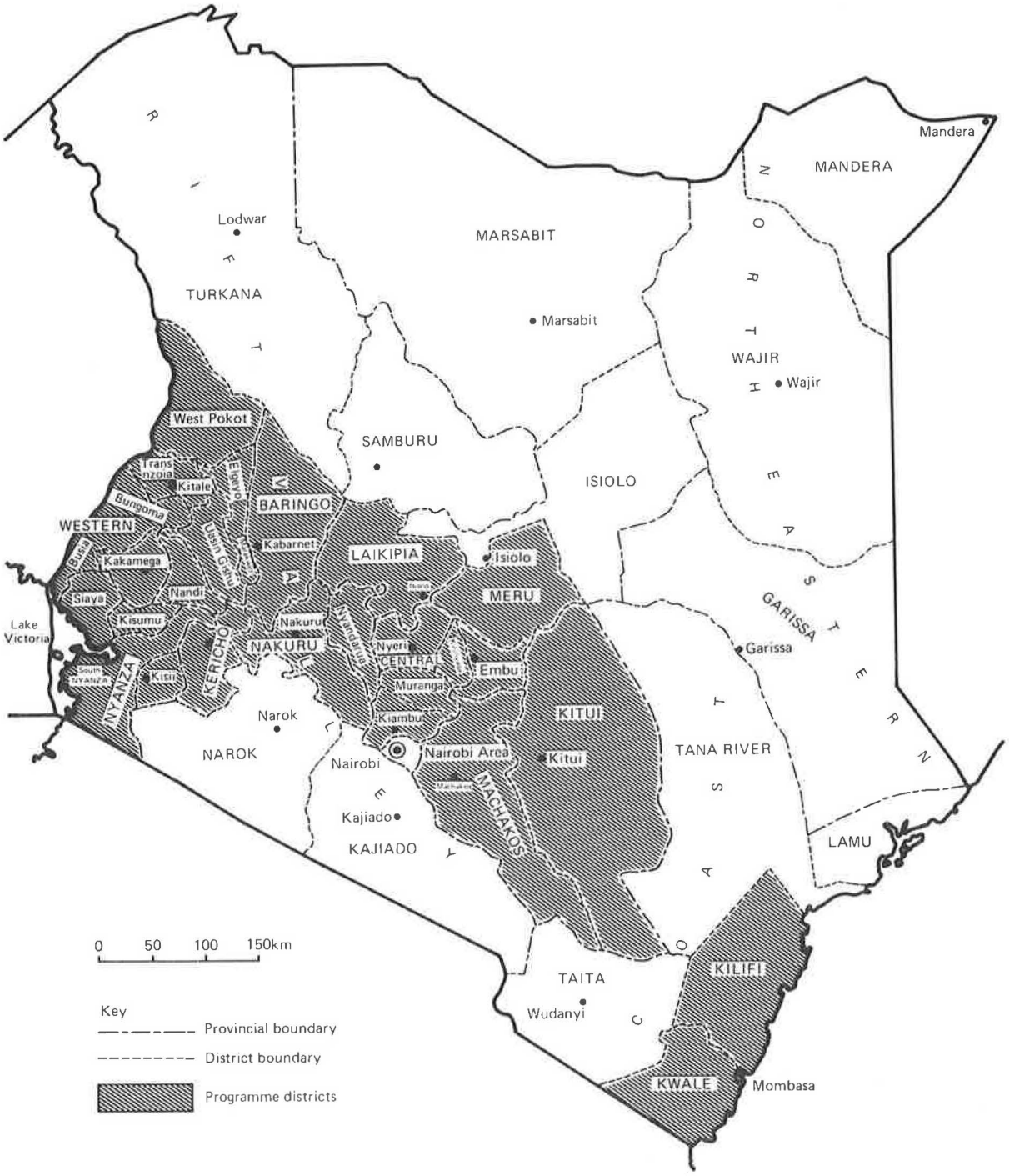


FIGURE 13 Kenya MRP districts.

maintenance activity and some of the activities had to be broken into two-component operations for ease of control or measurement.

Forms were developed to record the relevant technical and personnel data for each day's task. They included name, sex, and age of individuals. Reports were thoroughly checked before being entered on the microcomputer data base, using dBase III Plus software.

Productivities were calculated in terms of quantity of work completed per hour. These values were then adjusted to represent the quantity of work that could be expected for a standard 5-hr closely supervised workday. This work was adopted as the basis for development of daily productivity standards. In the normal situation, the lengthman is subjected to a lower level of supervision. He or she is nominally paid for 8 hr of work, and the 5-hr supervised standard was considered to be an appropriate target for the MRP.

From the data gathered, comparisons were made between performance related to age groups, sex, and districts, and whether the set tasks were successfully completed or not. The lengthmen were given individual daily tasks on various maintenance activities along a short section of road. Tasks were set daily by the overseers and monitored throughout the day by both the overseers and headmen. The starting and finishing times were carefully recorded for each task.

During the study, the contractors were retained for approximately 7 hr if tasks were not completed. However, they were released earlier, normally after 5 hr, if they finished their work satisfactorily. Individuals were rotated between activities on successive days.

Phase I Study Results

A total of 1,960 task-days of data was collected. Of the 11 originally planned activities, 5 were subdivided into two operations, in effect making 16 activities to be monitored, as follows:

Activity Code	*Maintained Activity*
R1	Inspection and removal of obstructions
R2A	Clean culverts and inlets
R2B	Clean culvert outfalls
R3	Repair culvert headwalls
R4	Clean miter drains
R5	Clean side drains
R6A	Repair scour checks
R6B	Repair side drain erosion
R7A	Repair shoulder erosion
R7B	Grass planting
R8A	Fill potholes in carriageway
R8B	Fill ruts in carriageway
R9A	Grub edge of carriageway
R9B	Reshape carriageway
R10	Grass cutting
R11	Bush clearing

For each activity, up to four degrees of difficulty were defined and observations made on the basis of these divisions. As many as 47 observations were made on an individual activity (Activity R3). Records were unobtainable for only one combination of activity and difficulty. For only four activity-difficulty combinations were five or fewer observations made. The lack of data on these combinations reflects their low occurrence in general RA/MR maintenance operations.

The observations were taken on seven road sites, two of which were constructed to minor road cross section standards. From the observations taken, it was not possible to detect a difference in productivity between rural access roads and minor roads.

Little difference was observed in productivities between age groups, districts, or sexes. The most significant observation was that contractors who completed their tasks, and thus left work early, worked significantly harder (by up to 30 percent) than those who failed to do so. This trend shows up fairly consistently throughout the results and is (a) an indication of the quality of the data gathered and (b) a strong argument for establishing a task-based system with fair and achievable targets.

The standard deviation for each data set was calculated and used to define the representative range of productivity for the task around the mean of the observations. These ranges were then used for the final comparisons and are presented in Table 2 (Kisii district) and Table 3 (South Nyanza district).

The productivity results presented in these tables can be used as standards for the MRP. However, these results were obtained under most favorable supervision conditions with trained lengthmen and motivated overseers. They should be treated as reference points to strive for by MRP personnel.

PHASE II: IDENTIFICATION OF ROUTINE MAINTENANCE REQUIREMENTS

Objectives

The principal objectives of Phase II of the study are as follows:

1. To determine the influence of geometry, climate, traffic, and pavement materials on the amount of routine maintenance required expressed in terms of the 16 standard activities; and
2. To determine optimum maintainable lengths of road relative to labor inputs under various soil, topographic, climatic, and traffic conditions.

Methodology

In order to achieve these objectives, detailed observations will be taken over a period of 12 months on existing sections of RA/MRs that are already under routine maintenance.

Selection of Road Sections and Preparation Work

The Kenyan RA/MRs have been constructed over a wide range of climates, vertical gradients, soil types, and traffic levels. Traffic currently using these roads varies from less than 10 to over 100 vpd. Selection will be made of suitable sections of existing roads encompassing as many of these variables as possible.

The most important variables considered to affect the performance of the roads are classified as follows:

- Traffic <20, 20 to 50, and >50 vpd;
- Annual rainfall <500, 500 to 1 500, and >1 500 mm;

TABLE 2 PRODUCTIVITY RANGES FOR ROUTINE MAINTENANCE IN WET HIGHLAND AREAS (KISII DISTRICT)

Activity	Unit	Task Difficulty 1	2	3	4	Notes
R2A: Clean culvert and inlet	As shown	2–5 culverts/day	1–1½ days/culvert	1½–3 days/culvert	3–5 days/culvert	Difficulty—silt depth in culvert 1, up to ¼; 2, ¼ to ½; 3, ½ to ¾; 4, over ¾
R2B: Clean culvert outfalls	m/day	45–60	30–45	15–30	–	Difficulty—silt depth 1, up to 10 cm; 2, 10 to 20 cm; 3, over 20 cm
R3: Repair culvert headwalls	No./day	5–10	3–5	–	–	Difficulty—type of repair 1, minor repairs; 2, major repairs
R4: Clean miter drains	m/day	50–65	35–50	25–35	–	Difficulty—silt depth 1, up to 10 cm; 2, 10 to 15 cm; 3, over 15 cm
R5: Clean side drains	m/day	55–75	35–55	25–35	–	Difficulty—silt depth 1, up to 10 cm; 2, 10 to 15 cm; 3, over 15 cm
R6A: Repair scour checks	No./day	3–8	5–9	–	–	Difficulty—type of scour check 1, wood; 2, stone
R6B: Repair side drain erosion	m/day	90–110	70–90	50–70	–	Difficulty—depth of erosion 1, up to 15 cm; 2, 15 to 30 cm; 3, over 30 cm
R7A: Repair shoulder erosion	m/day	85–110	60–85	40–60	–	Difficulty—depth of erosion 1, up to 10 cm; 2, 10 to 15 cm; 3, over 15 cm
R7B: Grass planting	m/day	90–110	70–90	60–70	–	Difficulty—planting width 1, up to 0.5 m; 2, 0.5 to 1.0 m; 3, over 1.0 m
R8A: Fill potholes in carriageway	w.brws/day	23–30	16–23	11–16	7–11	Difficulty—hauling distance 1, no haul; 2, up to 100 m; 3, 100 to 200 m; 4, over 200 m
R8B: Fill ruts in carriageway	m/day	60–80	40–60	30–40	10–20	Difficulty—hauling distance 1, no haul; 2, up to 100 m; 3, 100 to 200 m; 4, over 200 m
R9A: Grub edge of carriageway	m/day	230–300	160–230	100–160	–	Difficulty—width of grubbing 1, up to 0.5 m; 2, 0.5 to 1.0 m; 3, over 1.0 m
R9B: Reshape carriageway[a]	m/day	55–80	40–55	–	–	Difficulty—type of reshaping 1, light (up to 75 mm); 2, heavy (over 75 mm)
R10: Grass cutting						
Light	m/day	400–450	220–300	150–220	–	Difficulty—width of grass cutting 1, up to 1.0 m; 2, 1.0 to 2.0 m; 3, over 2.0 m
Dense	m/day	275–350	200–275	150–200	–	
R11: Bush cutting						
Light	m/day	400–450	220–300	150–220	–	Difficulty—width of bush 1, up to 1.0 m; 2, 1.0 to 2.0 m; 3, over 2.0 m
Dense	m/day	250–300	200–250	150–200	–	

[a]All tasks except reshaping are measured along one side of the road only.

- Vertical gradients <4, 4 to 8, and >8 percent; and
- Surface materials, graveled and ungraveled.

In addition, the following parameters will also be recorded at the beginning and end of the study:

- Width of carriageway,
- Average crossfall,
- Length of section maintained,
- Frequency of supervision, and
- Thickness of gravel.

As a control, some sections will be selected on which no maintenance (other than emergency repairs) will be carried out during the study.

MRP personnel will be responsible for the initial identification of roads suitable for inclusion in the study. A total of about 50 sections of roads will be required to cover all the principal variables (though some sections may be on the same road). Each section will be of a length maintainable by one individual contractor, i.e., 1 to 2 km, approximately.

The study roads will mainly be concentrated in three areas, Nakuru/Naivasha, South Nyanza, and Kisii; to cover a wider range of variables, some road sections may be located outside these areas.

Each road will be inspected by a supervising engineer to confirm its suitability and to carry out a detailed condition survey. This will include identifying maintenance defects and preparing a schedule of repairs.

The supervising engineer will also be responsible for the final demarcation of the road sections to be included in the study, and will also ensure that each road section is clearly marked with a signboard indicating the section reference number. This initial survey will record the relevant details of soil

TABLE 3 PRODUCTIVITY RANGES FOR ROUTINE MAINTENANCE IN DRY LOWLAND AREAS (SOUTH NYANZA DISTRICT)

Activity	Unit	Task Difficulty 1	2	3	4	Notes
R2A: Clean culvert and inlet	As shown	2–5 culverts/ day	1–1½ days/ culvert	1½–3 days/ culvert	3–5 days/ culvert	Difficulty—silt depth in culvert 1, up to ¼; 2, ¼ to ½; 3, ½ to ¾; 4, over ¾ Tasks for 600 dia culverts with 7 rings
R2B: Clean culvert outfalls	m/day	45–60	30–45	15–30	–	Difficulty—silt depth 1, up to 10 cm; 2, 10 to 20 cm; 3, over 20 cm
R3: Repair culvert headwalls	No./day	5–10	3–5	–	–	Difficulty—type of repair 1, minor repairs; 2, major repairs
R4: Clean miter drains	m/day	50–65	35–50	25–35	–	Difficulty—silt depth 1, up to 10 cm; 2, 10 to 15 cm; 3, over 15 cm
R5: Clean side drains	m/day	soft, 45–60; hard, 25–35	soft, 35–45; hard, 20–25	soft, 25–35; hard, 20–25	–	Difficulty—silt depth 1, up to 10 cm; 2, 10 to 15 cm; 3, over 15 cm
R6A: Repair scour checks	No./day	3–8	5–9	–	–	Difficulty—type of scour check 1, wood; 2, stone
R6B: Repair side drain erosion	m/day	75–100	40–75	20–40	–	Difficulty—depth of erosion 1, up to 15 cm; 2, 15 to 30 cm; 3, over 30 cm
R7A: Repair shoulder erosion	m/day	85–110	60–85	40–60	–	Difficulty—depth of erosion 1, up to 10 cm; 2, 10 to 15 cm; 3, over 15 cm
R7B: Grass planting	m/day	90–110	70–90	60–70	–	Difficulty—planting width 1, up to 0.5 m; 2, 0.5 to 1.0 m; 3, over 1.0 m
R8A: Fill potholes in carriageway	w.brws/day	20–30	14–20	10–14	7–10	Difficulty—hauling distance 1, no haul; 2, up to 100 m; 3, 100 to 200 m; 4, over 200 m
R8B: Fill ruts in carriageway	m/day	40–60	20–40	10–20	5–10	Difficulty—hauling distance 1, no haul; 2, up to 100 m; 3, 100 to 200 m; 4, over 200 m
R9A: Grub edge of carriageway	m/day	150–220	80–150	50–80	–	Difficulty—width of grubbing 1, up to 0.5 m; 2, 0.5 to 1.0 m; 3, over 1.0 m
R9B: Reshape carriageway[a]	m/day	55–80	40–55	–	–	Difficulty—type of reshaping 1, light (up to 75 mm); 2, heavy (over 75 mm)
R10: Grass cutting						
Light	m/day	275–350	200–275	150–200	–	Difficulty—width of grass cutting 1, up to 1.0 m; 2, 1.0 to 2.0 m; 3, over 2.0 m
Dense	m/day	275–350	200–275	150–200	–	
R11: Bush cutting						
Light	m/day	400–450	220–300	150–220	–	Difficulty—width of bush 1, up to 1.0 m; 2, 1.0 to 2.0 m; 3, over 2.0 m
Dense	m/day	250–300	200–250	150–200	–	

[a]All tasks except reshaping are measured along one side of the road only.

and gravel type, mean gradient, gravel thickness, road width, etc., for each contractor's section.

MRP personnel will be responsible for carrying out the repairs to bring the road to an acceptable and maintainable standard within a reasonable period of time to suit the study program. A check inspection on completion of the repair works will be carried out by the supervising engineer.

Before the commencement of the study and at the end of the study, manual traffic counts will be taken over seven consecutive days and two nights on each road and thereafter on 1 day per month.

Rainfall gauges will be established in the vicinity of each road to monitor daily rainfall, and surface roughness measurement will be carried out on each road section using the MRP vehicle-mounted bump integrator on a monthly basis.

Study Data Collection

The study will be carried out over a period of 12 consecutive months on each road section with an additional 1 month's trial data collection at the commencement of the study.

Every month, an MRP inspector, under the direction of the supervising engineer, will visit each road and record and assess the previous month's work. He will also survey the road and locate, quantify, and record all defects or outstand-

ing maintenance requirements. He will inform the overseer supervising the road of the maintenance program for the following month on the basis of the productivity norms established by the MRP under *Maintenance Study II Phase I Productivity Standards* (*4*), which was completed in early 1989.

Each month the schedule for each contractor will show (a) maintenance work achieved and (b) outstanding work required.

If the maintenance contractor is falling behind with the work, then he will be given additional time to ensure that the road condition is satisfactorily maintained (provided the delays are not caused by inefficiency on the part of the contractor). In certain cases, additional personnel may be temporarily employed to help catch up on any backlog of work.

All survey data including the regular monitoring of rainfall, roughness, and traffic will be input to a microcomputer data base and analyzed to establish relationships between the extent of each maintenance operating required and the monitored characteristics of the road section.

PROJECT OUTPUT

The two phases of the study should enable the labor-based maintenance of RA/MRs to be significantly improved.

The research in Phases I and II will result in

1. A rational basis for implementing routine maintenance using the lengthman principle over the range of site conditions experienced in Kenya,
2. Identification of any shortcomings in the lengthman system,
3. The provision of data to prepare realistic estimates of resources and costs for routine maintenance, and
4. More realistic productivity standards for use in the TRRL guide to maintenance management for district engineers (*5*).

These results will permit more effective planning, execution, and monitoring of gravel road maintenance, particularly when lengthmen contractors are used.

OTHER INITIATIVES

In addition to the Phase I and Phase II studies, the Ministry of Public Works has taken a number of other initiatives to improve the lengthman maintenance system.

The improvements identified and described earlier have been adopted as policy for the MRP. Pending the results of the two study phases, including completion of productivity standards and maintenance needs, other improvements will be introduced through a planned program of training and demonstration sites.

In recent TRRL studies, research on unpaved roads has often concentrated on comparisons between motorized graders and tractor-drawn graders (*6*).

The maintenance project in Kenya has afforded the opportunity of evaluating maintenance strategies incorporating a mix of towed graders and contract lengthmen. The intention is to investigate a mix of low-cost equipment and labor-based maintenance during Phase II of the project. This strategy will be used only on the more highly trafficked roads (>50 vpd) of the Minor Roads Program. It is intended that the towed grader will only lightly grade the running surface, with pothole patching and all side drainage and off-carriageway work being carried out by the lengthmen.

The results from this additional research will provide new and valuable information on the most appropriate maintenance strategies using low-cost techniques for gravel roads in developing countries.

RECOMMENDATIONS FROM THE PHASE I INVESTIGATION

The results of Phase I of the study are summarized in Tables 2 and 3. For the guidelines to be applicable in each district of Kenya, some allowance, based on previous experience, will be required to adjust the separate task rates for the particular workforce involved.

The subdivision of five of the maintenance activities permits better control and measurement of these tasks and has been recommended for incorporation into the present reporting system.

The guidelines themselves have been established under virtually ideal supervisory conditions with a well-trained workforce under the control of motivated overseers. As such, the guidelines represent the best overall productivities that can be expected under conditions in Kisii and South Nyanza districts. They should be treated as realistic targets that can be strived for as the management of routine maintenance is improved. More important perhaps, the guidelines provide a reference against which the performance of contractors can be assessed.

The maintenance activities are not directly comparable to those of construction and improvement work. A number of activities are of a similar nature although there are differences in scale of work and in methods of working and measurement. When a broad comparison is possible, the productivities achieved for maintenance activities are somewhat below the RARP/MRP construction productivity standards. This fact reflects the generally smaller scale and dispersed nature of the maintenance work.

At the commencement of Phase I, it was found that many contractors and headmen did not fully appreciate what the various maintenance tasks were, how they should be carried out and controlled, or what methods were to be used for measuring them. There was an obvious need for training, particularly because, if the task system was to work effectively, the headmen had to take on a much stricter supervisory function than they had in the past. Besides their direct supervisory function, headmen had to also become fully conversant with the methods for measuring each activity so that they could assist the overseers in their overall management role.

It will be important to introduce the experience gained from Phase I, and in due course that from Phase II, into the training and maintenance operations of the MRP.

The lengthmen, headmen, and overseers should have a clear understanding of the reasons for and methods of desilting. It is just as important that they appreciate when desilting is not required because of established vegetation and good drainage conditions.

Recommendations include updating of the planning and reporting system and introduction of the new productivity standards. Training material, manuals, and courses will also be necessary.

ACKNOWLEDGMENTS

The authors wish to acknowledge the support given to the research project by S. Otonglo; the staff of the Ministry of Public Works, especially that of B. Ariga; and the Swiss Development Corporation, who funded Phases I and II of the project.

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Views expressed are not necessarily those of the British government.

Use of Freight Tariffs to Predict Road Investment Benefits: Some Lessons from Pakistan

J. L. HINE AND A. S. CHILVER

There are several standard models of vehicle operating costs (VOCs) that are advocated for the appraisal of rural road investment in developing countries. The World Bank's Highway Design and Maintenance Standards Model (HDM) and the Transport and Road Research Laboratory's Road Transport Investment Model (RTIM) incorporate different models of VOCs based on research carried out in Kenya, the Caribbean, Brazil, and India. In the HDM, the user can select the most appropriate VOC model to suit local conditions. Because the different models give different estimates of benefits, a convenient way of selecting the most appropriate model form and of calibrating the results is needed. The role of competitively determined freight tariffs to assist with this task was investigated using data from rough and smooth roads in Pakistan. Although there are particular problems with using freight rates such as seasonality, empty running, and traffic directional flow imbalance, the results show that freight rates could prove to be useful in checking models of VOCs. Vehicle maintenance costs in Pakistan are particularly low, and therefore all of the standard models overestimated the difference in operating costs (per vehicle-kilometer) between rough and smooth roads. However, vehicle load weights were found to be lower on rough roads than on smooth roads. If loads increased with road improvement, important additional benefits would result that could be easily overlooked.

Savings in vehicle operating costs (VOCs) have long been identified as a major benefit arising from the improvement of low-cost roads. In the economic appraisal of road investment, savings in VOCs are predicted to follow from reduced surface roughness and better road alignment. However, it can be expensive and difficult to collect direct evidence to quantify how changes in road surface characteristics will directly affect VOCs.

The principal method now advocated to appraise the investment and maintenance of rural roads in developing countries is to use appraisal models such as the Road Transport Investment Model (RTIM) (*1*) or the Highway Design and Maintenance Standards Model (HDM) (*2*), which were developed by the Transport and Road Research Laboratory (TRRL) of the United Kingdom and by the World Bank, respectively. Within these appraisal models, different sets of VOC relationships have been derived from research carried out in Kenya, the Caribbean, Brazil, and India. Because of the diversity of results, one model form will not satisfactorily encapsulate VOCs for all developing countries. In the HDM, different VOC models are provided that enable the user to select the most appropriate one to suit local conditions. In addition, there is some limited provision for calibrating the model chosen.

In view of the problems of selecting an appropriate model of VOCs, there is a need for additional information to assist in this choice. Where they are available, competitively determined freight tariffs could provide a useful way of selecting, checking, and helping to calibrate the VOC model.

An underlying hypothesis of this approach is that freight tariffs reflect, to a reasonable degree, the total operating costs of freight vehicles. For most countries, there are grounds to support this point of view. Private operators usually form an important component of the freight transport industry, and freight transport is, on the whole, subject to far less government control than passenger transport. Unlike passenger transport, subsidies are less prevalent and freight tariffs are less controlled than passenger fares. Even where controls exist, tariffs should bear some relation to total costs.

As part of a major study of freight transport in Pakistan, it was decided to investigate how freight tariffs vary between rough and smooth roads and examine how tariffs match empirically determined VOCs and those predicted by the HDM and RTIM models. Most of the field work for this study was collected during 1986.

A number of different surveys were undertaken for the study. The main element was a roadside interview survey in which 3,500 truck drivers were stopped and interviewed at 39 sites throughout the country. Because truck drivers are responsible for finding work, collecting revenue, keeping accounts, and maintaining their vehicles, they were able to answer a wide range of questions on the vehicles' operations. Information was collected on vehicle type, journeys, loads, tariffs, costs, vehicle finance, operating performance, and fleet management.

Nearly all of Pakistan's main roads are paved, although earth and gravel roads make up just over half of the total network. Most of the roadside survey sites were located on the main paved roads. To ensure that sufficient data were collected from the unpaved network, five survey sites were chosen covering the large unpaved road network of the Mekran area of south Baluchistan.

Cost and revenue data were also collected from a study of 50 drivers' diaries, which, in some cases, recorded their operations for a period of many years. Other surveys were undertaken covering freight agents, freight consigners, and vehicle use.

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In order to identify trends and seasonality in tariffs, past tariff data were collected from records of freight agents relating to particular journeys with standard loads. In total, 120 different series of data were collected. Most of these data extended over a period of 4½ years.

To supplement road roughness data already collected by consultants for most of the main road network, an additional survey of road roughness was undertaken using a vehicle-mounted bump integrator unit that was calibrated with a Merlin, TRRL's low-cost roughness measuring instrument. This survey covered the rough unpaved roads of Baluchistan and part of the secondary road network.

ROAD FREIGHT TRANSPORT IN PAKISTAN

In 1983 it was estimated that there were 45,000 privately registered trucks in Pakistan (*3*). In the roadside survey, two-axle Bedford trucks, with 98-hp engines and a design carrying capacity of 7 tons, accounted for 77 percent of the vehicles surveyed. Two-axle Japanese trucks, with a design capacity of about 12 tons and engines in the range of 140 to 220 hp, accounted for another 14 percent. Three-axle vehicles and tractor-trailers made up another 7 percent.

In recent years, Japanese trucks have taken a growing proportion of the market. Most trucks in Pakistan are strengthened after assembly to take heavier loads. The most popular modifications are the strengthening of the chassis, axle springs, and engine compartment. Wheel rims, tires, and axles are also changed for heavier-duty items. Bedford trucks commonly carry 11 tons and the two-axle Japanese trucks carry 16 tons. Some two-axle Japanese trucks are converted to three axles carrying up to 30 tons, whereas others are converted to tractor units with the addition of a fifth wheel. The chassis of semitrailers are strengthened and loads of 55 tons are not uncommon.

Vehicle bodies are made locally and fitted to the truck chassis after they have left the assembly plant. The vehicles are highly decorated and there is usually a purpose-built space on top of the cab where the drivers and assistants can rest or sleep while the vehicle is in motion.

Some variation was found in the geographical spread of the different vehicle types. Two-axle Japanese trucks were found to be much more popular in Baluchistan than in the rest of the country, where the traditional Bedford truck appears to be more popular. On the unpaved roads of the Mekran area of Baluchistan, hardly any three-axle vehicles were recorded by the survey and articulated vehicles were entirely absent. Apart from their need in the servicing of the petroleum and coal mining industries, large-capacity vehicles tend to be restricted to the paved main roads.

About 95 percent of the trucking fleet is privately owned. There are few large private transport operators and only a limited number of industrial firms operate "own account" trucking fleets; they prefer to rely on hiring transport. The industry is mainly composed of a large number of individual owners operating just one vehicle that provide a "hire and reward" service. The pattern of numerous small-scale entrepreneurs is also a feature of the ancillary services of the industry such as freight agents and vehicle workshops. All of these are spread throughout the country.

Vehicle modifications and repairs are carried out by groups of small workshops. Each workshop has relatively little working space, employs no more than a handful of people, and has only a limited access to machinery. It tends to specialize in providing a particular service, but in most towns a wide range of skills and machine tools is available and usually a full range of repairs can be carried out.

Spare parts for Bedford trucks are cheap and widely available. Many of them are made in Pakistan and, in most cases, if not immediately available, they can be made locally. In the larger towns, imported spares for the Japanese trucks are available, but they tend to be about three times the price of the equivalent part for the Bedford.

Road freight transport in Pakistan is largely organized on a free market basis; freight tariffs are competitively determined by supply and demand, and there is little direct government intervention in the industry. Entry into the industry is cheap and easy. There is a relatively lax licensing system, and there is little enforcement of axle load limits or of vehicle construction and use regulations.

Because most loads are placed through freight agents, they play a pivotal role in the operations of the industry. Freight agents are also involved in running warehouses and in providing finance for buying and selling trucks.

Truck drivers will travel from job to job for up to 4 weeks at a time before returning to base. Although there is some degree of regional specialization, if necessary, drivers will travel long distances to other provinces in search of work. Trip distances of over 1 000 km are common. Most trucks carry two drivers to allow for working around the clock and an assistant to look after the load and deal with the payment of local taxes at the district octroi posts. Although some drivers own part or all of the vehicle, most are employees. The principal driver is responsible for finding loads, collecting revenue, keeping accounts, and maintaining the vehicle.

Vehicle running speeds are low; average spot running speeds have been estimated to be 52 kph (*4*), whereas overall long-distance journey speeds (including rest periods) were found to be about 23 kph. Despite this fact, vehicles are used intensively. The survey of vehicle use found that vehicles were in active use (i.e., moving, loading, or unloading) for over 12 hr per day and it appears that they travel over 100 000 km per year.

TRUCK OPERATING COSTS AND PROFITABILITY FOR THE MAIN ROAD NETWORK

A substantial amount of data on operating costs, revenues, and vehicle use was collected by the different surveys for vehicles traveling on the main road network. From these data, it was possible to estimate mean lifetime operating costs and revenues for the Bedford and two-axle Japanese trucks. At the time of the surveys, the mean age of Japanese vehicles was comparatively low; adjustments had to be made using trends in the data (particularly with regard to maintenance costs and annual distance traveled) to compensate for this.

The mean road roughness for the main road network was found to be 4800 mm/km using the bump integrator (BI)

unit. This is equivalent to 6.0 m/km on the International Roughness Index (IRI).

The data presented in Table 1 are estimates of long-term costs and revenues in Pakistan rupees per kilometer (R/km), for the two types of vehicle. For both vehicle types, revenues were closely related to long-term costs. The two-axle Japanese trucks appear to be reasonably profitable, whereas the profitability of Bedford trucks appears to be marginal.

ROUGH AND SMOOTH ROUTES

Comparison of Costs

To investigate the influence of road roughness on costs and tariffs, a comparison was made between data collected from the main paved network and data from the route between Karachi and Turbat in the Mekran. About 30 percent of road length on this route (from Karachi to Bela) consists of a paved road of a reasonable standard, and the remaining 70 percent is mostly composed of a rough two-lane track. The surface of the track is mostly sand interspersed with rock outcrops. The average roughness level for the whole route was estimated to be 12 900 mm/km (BI) or 14.6 m/km (IRI) in comparison with an average roughness level of 4800 mm/km (BI) or 6.0 m/km (IRI) for the main paved network.

Data were collected from 68 freight vehicles operating on the rough route in the Mekran; 54 of these vehicles were loaded. Nearly all the vehicles appeared to use this route frequently; 90 percent of the drivers came from the Mekran and most returned home to their family at least every 2 weeks. However, it was not determined to what extent the vehicles had been used on this or on other routes. From the data, some key differences in operating costs were identified; these are presented in Table 2.

The trucks operating on the rough roads in the Mekran were shown to have higher fuel and tire consumption compared with those operating on the smoother routes. A substantial difference in maintenance costs was found for the Japanese trucks, although there was little apparent difference

TABLE 1 ESTIMATED LIFETIME OPERATING COSTS AND REVENUES FOR VEHICLES RUNNING ON MAIN ROAD NETWORK (R/km)

	Two-Axle Bedford	Two-Axle Japanese
Fuel	1.257	1.333
Oil and grease	0.141	0.190
Tires	0.142	0.142
Maintenance (parts and labor)	0.322	0.294
Crew	0.426	0.472
Depreciation and interest	0.243	0.365
Loading labor	0.079	0.086
Octroi, police, taxes	0.171	0.193
Agents' commission	0.078	0.115
Total costs	2.859	3.190
Lifetime revenue	2.939	3.347
Net Profit, per km	0.080	0.157
Calculated IRR, %	6	15

(1986 prices, approximate exchange rate: Pak Rs 16.6 = US $ 1)

TABLE 2 DIFFERENCES IN OPERATING COSTS IDENTIFIED FOR TRUCKS TRAVELING ON ROUGH COMPARED WITH SMOOTH ROADS

	Two-Axle Bedford (% change)	Two-Axle Japanese (% change)
Fuel	+20	+19
Maintenance	- 2	+41
Tires	+36	+58
Crew Costs	-10	-11
Annual Distance Travelled	- 3	- 5

for the Bedford trucks. The lower crew costs recorded on the rough roads in the Mekran were mainly because of a lower proportion of vehicles with two drivers.

It was not possible to identify any differences in expected vehicle lifetimes for trucks operating on the rough and smooth roads, although trucks operating on the rough roads were marginally younger.

Comparison of Tariffs

In response to the pressures of supply and demand, it was found that tariff rates varied from day to day and from commodity to commodity. However, consistent patterns were found relating to trip distance, direction, and seasonality. As expected, vehicles traveling on shorter journeys were found to have higher rates than those traveling on longer journeys. The tariff rates of vehicles traveling away from Karachi were found to be consistently higher throughout the whole country than for those vehicles traveling in the opposite direction. This reflected the flow imbalance in which relatively more freight moves inland from the port of Karachi than in the reverse direction. In order to compare freight tariffs on rough and smooth roads, it was necessary to take explicit account of these two factors.

An analysis of trends in past tariff data for the whole country showed that tariff rates closely followed the rate of inflation. A small but consistent degree of seasonality was also found to be present. In the first half of the year, a deviation in tariff rates of about +0.6 percent from the trend was found; this was followed by a deviation of about −1.0 percent in the second half of the year. No marked differences were found in the trends of past tariff rates covering the Mekran area compared with data for the rest of Pakistan, and there was no evidence to suggest that rates during the survey period were in any way atypical. As a result of this analysis, no adjustments were made for seasonality in the comparison of tariffs on rough and smooth roads.

Trip distances on the Mekran route varied between 600 and 900 km. For the comparison, data on journeys of between 500 and 1 000 km in length were selected from the roadside survey. In total, data on 330 Japanese and Bedford two-axle trucks were analyzed for the smooth route comparison.

From the data in Table 3, mean values of the expected revenue per kilometer and per ton-kilometer were calculated for a standardized complete round trip for each vehicle type. In the estimation of revenue per kilometer, empty running was taken into account using the loaded ratios found in the survey. The results are presented in Table 4.

TABLE 3 TRIP SURVEY DATA

Route		Rough		Smooth	
To/From Karachi		To	From	To	From
Two-axle Bedford trucks:-					
Loaded trucks		3	12	105	132
Empty trucks		5	0	14	8
Loaded ratio	%	37.5	100	88.2	94.3
Mean loaded distance	km	781	726	726	698
Mean empty distance	km	627	-	719	751
Mean load	tons	5.5	8.3	8.0	8.7
Mean tariff	Rs	900	4617	1980	2680
Two-axle Japanese trucks:-					
Loaded trucks		8	31	15	42
Empty trucks		9	0	12	2
Loaded ratio	%	47.1	100	55.5	95.5
Mean loaded distance	km	876	649	748	847
Mean empty distance	km	676	-	667	704
Mean load	tons	7.2	9.4	11	13
Mean tariff	Rs	2825	5597	3014	4381

TABLE 4 STANDARDIZED TRIP DATA

		Two-Axle Bedford		Two-Axle Japanese	
Route		Rough	Smooth	Rough	Smooth
Road roughness	mm/km	12900	4800	12900	4800
Standardised distance	km	754	713	763	798
Expected load	tons	5.2	7.7	6.4	9.3
Expected revenue	Rs	2477	2136	3464	2930
Expected rev/km	Rs	3.29	3.0	4.54	3.67
Expected rev/ton km	Rs	0.637	0.390	0.712	0.397

Table 4 indicates that revenue per kilometer was recorded to be 10 and 24 percent higher on the rough routes than on the smooth routes for Bedford and Japanese trucks, respectively. Revenue per ton-kilometer was found to be 63 and 79 percent higher on the rough route compared with the smooth route for Bedford and Japanese trucks, respectively.

COMPARISONS WITH VOC MODEL PREDICTIONS

Three sets of relationships were used in the comparisons of tariffs with VOC model predictions. The relationships derived from studies in Kenya and the Caribbean by the TRRL are incorporated into the amended version of the Road Transport Investment Model, RTIM2 (*1*). The relationships derived from studies in Brazil and India are incorporated as separate options in the latest version of the World Bank's HDM3 (*2*).

Key input assumptions relating to the models are presented in Table 5. For consistency with revenue data, market prices (including taxes and duties) are used for all items. The VOC models predict maintenance parts costs as a percentage of the new-vehicle price where both parts and vehicles are priced without tax. Because the average tax rates on vehicles and parts were believed to be broadly similar, no adjustments were made to equate the predicted maintenance parts costs to market price levels.

For some of the VOC model relationships, it is necessary to specify pavement type with roughness. For simplification, the rough and smooth routes were classified as unpaved and paved, respectively.

Data on gradients and curvature were not collected; road roughness was the only difference between rough and smooth roads examined by the models. Apart from the key input assumptions, advisory default options were used. No attempt was made to adjust or calibrate any of the model relationships used.

The results of the model predictions are shown in Tables 6–8. The tables indicate large differences between the different models in fuel consumption and maintenance costs. In the RTIM2 relationships, fuel consumption is very sensitive to gradient, and clearly a major part of the difference in fuel consumption between the models relates to this. Because of the lack of reliable information on gradients, it was decided to omit fuel consumption from the comparison.

Nearly all the main paved roads in Pakistan are flat and straight. The Mekran-Karachi road passes through two ranges of hills, and naturally some gradients and curvature are present. Rough calculations suggest that had an allowance been made for gradient and curvature, tire consumption for the Brazilian and Indian VOC estimates would have been about 5 percent higher on the rough route. Likewise, parts consumption would be about 2 percent higher on the rough route for the RTIM2 VOC estimates.

Fuel consumption data collected from the roadside interview survey on the rough and smooth routes are as follows:

Truck	*Rough (R/km)*	*Smooth (R/km)*
Two-axle Bedford	1.483	1.233
Two-axle Japanese	1.534	1.290

These data are deducted from the revenues presented in Table 4 to give the observed revenues, without fuel consumption,

TABLE 5 KEY ASSUMPTIONS FOR VOC MODEL PREDICTIONS

		Two-Axle Bedfords	Two-Axle Japanese
All VOC Models:-			
Annual travel	km	112,000	123,000
New vehicle price	Rs	325,000	390,000
Tire price	Rs	2,275	2,275
Diesel price	Rs/ltr	4.3	4.3
Oil price	Rs/ltr	13.4	13.4
Crew cost	Rs/hr	13	13
Maintenance labor	Rs/hr	9.4	9.4
Interest rate	%	10	10
Gradient	m/km	0	0
Curvature	%	0	0
Engine power	H P	98	180
RTIM2 Only:-			
Mean age	Years	9.3	3.4
Vehicle value /new price	%	25	62
Overhead rate	%	20	20
Gross weight (rough)	kg	10560	13380
Gross weight (smooth)	kg	13070	16250
HDM3 Only:-			
Mean life	Years	12	12
Tare weight	kg	5400	7000
Mean load (rough)	kg	5160	6380
Mean load (smooth)	kg	7670	9250

TABLE 6 RTIM2 VEHICLE OPERATING COST PREDICTIONS (R/km)

	Two-Axle Bedford		Two-Axle Japanese	
Road Type	Rough	Smooth	Rough	Smooth
Fuel	0.564	0.607	0.562	0.632
Oil	0.107	0.054	0.107	0.054
Tires	0.443	0.396	0.493	0.444
Maintenance parts	4.270	2.444	4.057	2.037
Maintenance labor	0.310	0.182	0.245	0.127
Crew	0.494	0.277	0.466	0.269
Depreciation	0.195	0.110	0.652	0.377
Interest	0.121	0.068	0.347	0.200
Overheads	1.310	0.822	1.393	0.819
Total	7.814	4.960	8.322	4.959

TABLE 7 HDM3 (BRAZIL) VEHICLE OPERATING COST PREDICTIONS (R/km)

	Two-Axle Bedford		Two-Axle Japanese	
Road Type	Rough	Smooth	Rough	Smooth
Fuel	1.344	1.345	1.499	1.618
Oil	0.071	0.053	0.071	0.053
Tires	0.353	0.341	0.371	0.364
Maintenance parts	4.033	1.717	3.450	1.469
Maintenance labor	0.233	0.150	0.196	0.126
Crew	0.376	0.210	0.372	0.192
Depreciation	0.303	0.185	0.356	0.203
Interest	0.182	0.111	0.214	0.122
Total	6.895	4.112	6.528	4.146

TABLE 8 HDM3 (INDIA) VEHICLE OPERATING COST PREDICTIONS (R/km)

	Two-Axle Bedford		Two-Axle Japanese	
Road Type	Rough	Smooth	Rough	Smooth
Fuel	0.966	1.240	0.817	1.133
Oil	0.043	0.037	0.043	0.037
Tires	0.397	0.293	0.360	0.272
Maintenance parts	1.069	0.382	1.003	0.366
Maintenance labor	0.745	0.310	0.634	0.268
Crew	0.290	0.290	0.264	0.264
Depreciation	0.141	0.141	0.325	0.325
Interest	0.096	0.096	0.221	0.221
Total	3.747	2.790	3.667	2.886

TABLE 9 COMPARISONS BETWEEN OBSERVED REVENUES AND PREDICTED COSTS, EXCLUDING FUEL (R/km)

	Two-Axle Bedford			Two-Axle Japanese		
	Rough Route	Smooth Route	Differ -ence	Rough Route	Smooth Route	Differ -ence
Observed rev/km	1.803	1.766	+ 0.037	3.007	2.382	+ 0.625
RTIM2 cost/km	7.250	4.353	+ 2.897	7.760	4.327	+ 3.433
HDM3(Brazil) cost/km	5.551	2.767	+ 2.784	5.029	2.528	+ 2.501
HDM3(India) cost/km	2.781	1.550	+ 1.231	2.850	1.753	+ 1.097

presented in Table 9. Table 9 also presents operating costs, after subtraction of fuel costs, predicted by the different models.

Table 9 indicates that wide variations exist between the different estimates of costs and revenues. As expected, the Indian relationships appear to provide the nearest estimates to the observed revenues. HDM3 (Brazil) and RTIM2 predict costs that are up to three and four times the observed revenues.

In Pakistan tariff revenues must cover both conventional operating costs and items such as loading and unloading labor, octroi charges (a local tax collected on vehicle movements), freight agents' fees, and gratuities paid to police. Typically, these costs account for about 10 percent of the tariff levied.

The data suggest that all of the models considerably overpredict the effect of increased road roughness on operating costs. This is particularly marked for RTIM2 and the Brazilian relationships. The Indian relationships for the two-axle Japanese vehicle provided the closest prediction of the differences in tariff levels between rough and smooth roads but, even in this case, the difference was overestimated by 43 percent.

Reductions in vehicle maintenance costs constitute most of the benefits predicted by the models following reduced levels of road roughness. In comparing the model predictions with estimated levels of maintenance costs derived from empirical data (see Table 1), it would appear that this particular component is substantially overestimated. The most obvious reasons for particularly low levels of vehicle maintenance costs in Pakistan probably relate to a number of factors, including the widespread network of skilled mechanics; the availability of low-cost, locally manufactured spare parts; and slow driving speeds. A factor that may also have a bearing on this is that the models suggest that oil changes may be far more frequent in Pakistan than in other countries.

Comparison of the Indian operating cost predictions with the predicted lifetime costs (Table 1) also shows important differences in tire consumption and crew costs. Tire costs appear to be overestimated, whereas crew costs are underestimated. Part of the explanation of the differences in tire consumption may relate to the slow running speeds and the widespread use of second-hand remolded tires in Pakistan. The differences in crew costs may relate to the generally high manning levels in Pakistan, although much of this difference could be overcome by changing the model input data.

DISCUSSION OF RESULTS

The evidence collected by all the surveys carried out in the study strongly supports the view that freight tariffs in Pakistan closely reflect the underlying costs of operating vehicles. Freight rates are competitively determined and, for the two-axle Bedfords and the Japanese vehicles, profits appear to be positive but not excessive. Freight rates are stable and most of the operators and drivers appear to be well experienced.

The analysis suggests that considerable care needs to be taken when road investment is appraised in countries not researched by the standard VOC models. Few road investment planners have the resources to mount their own detailed surveys of operating costs. Despite this fact, it would be sensible to collect some tariff data to see how closely the costs predicted by the models match revenues. A method of using tariff data to do this has been outlined.

Tariffs and revenues can easily help with the selection of the most appropriate model of vehicle operating costs. If circumstances permit, these data may also be used to calibrate the model of operating costs chosen; however, this would probably require some additional empirical cost data.

The HDM3 (Indian) relationships appear to provide the closest results to Pakistan's tariffs and operating costs. Unfortunately, a close examination of the cost and tariff data suggests that a successful modification of these relationships for road appraisal would be far from simple.

The analysis of costs and tariffs suggests that all three of the standard models of VOCs used to appraise rural road investment in developing countries would, without further modification, tend to overestimate the benefits of improving unpaved roads in Pakistan. This overestimate appears to be a consequence of the very low levels of vehicle maintenance costs that are found.

The loads carried by vehicles on the rough unsurfaced roads of Pakistan were about one-third lower than those on the main paved road network. This factor clearly needs some further investigation. If the improvement of the roads led to greater loads being carried by each vehicle, then there would be appreciable additional savings in operating costs.

ACKNOWLEDGMENTS

This work forms part of a program of joint research between the Overseas Unit (J. S. Yerrell, Head) of TRRL, United Kingdom, and the National Transport Research Centre (M. S. Swati, Head), Pakistan. It is published by permission of the director of TRRL.

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Views expressed herein are not necessarily those of the British government.

Microcomputer Optimization of Light-Pavement Rehabilitation

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A Markovian decision model on a personal computer is being developed for Finland's Roads Administration to optimize the allocation of funding among rehabilitation actions, geographic regions, and traffic volume classes on roads with oil gravel and other light pavements. In the long term, the model recommends a funding allocation and condition distribution that minimizes the sum of agency and user costs; in the short term, it recommends a strategy to maximize annual progress toward the long-term goal. As traffic volume and network length grow, the system simulates their effect on rehabilitation policy.

The Finnish National Roads Administration, with support from the consulting firms Viasys Oy of Finland and Cambridge Systematics, Inc., of the United States, is developing a national economic model to optimize light-pavement rehabilitation policy and the allocation of funding. The model is implemented as an extension to the administration's Highway Investment Programming System (HIPS) (*1*), which has been operational for all of the country's asphalt-surfaced roads for 2 years. It is able to analyze general rehabilitation procedures ranging from general patching to total reconstruction for all of the nation's 51,601 km of light pavements, evaluate the need for converting oil gravel pavements to asphalt concrete, and analyze the distribution of funding between the two types of pavement. Because it is a network-level model, the system analyzes policies at an aggregate level, considering only classes of roads, or subnetworks. A separate project analysis system (PAS) (*2*), now used in the district offices, relates the policy and budget recommendations of the network optimization to specific data bases of road segments.

Central to the optimization model is a Markov dynamic program, which has been formulated as a linear programming problem for solution by off-the-shelf software. The dynamic program categorizes pavements into 108 condition states and five actions, and represents deterioration as the probability of making transactions among all possible pairs of states over 1 year. An agency cost model estimates the cost of each possible action, and a user cost model evaluates the results in terms of travel time, fuel consumption, and vehicle depreciation. In selecting optimal actions for each possible condition state, the model tries to find a level of rehabilitation that balances the higher user costs of poor maintenance against the higher agency costs of good maintenance.

Separate models are available for each permutation of two climatic regions and three traffic volume classes; the Markov model optimizes budget allocations within each of these six subnetworks, and an incremental benefit-cost procedure based on parametric analysis results optimizes the allocation of funding among them and between light pavements and asphalt concrete.

A complete data set for the models has been prepared, and the necessary modifications to HIPS are nearing completion. As the section on performance and implementation issues will show, data collection efforts have been comparable in scope with any other pavement management methodology. Software performance on the microcomputers has been excellent for the asphalt models and is expected to be even better on the light pavement models. The system shows great promise for improving the economic basis for capital budgeting decisions on Finland's low-volume roads.

OVERALL MODEL STRUCTURE

The Finnish National Road Administration and the 13 district offices are highly independent in their day-to-day activities, with the central administration playing an administrative role and providing consulting services in new technologies and road and traffic research. Each year, the administration prepares rehabilitation and maintenance budgets and negotiates objectives with the districts and the Ministry of Transportation.

Annual and long-term road management objectives are set for each district, and these are strictly tied to the coming budget. The districts execute all maintenance actions, with the central office taking little interest in the specific actions chosen as long as overall objectives are met. Objectives are set for 1- and 5-year periods by mutual negotiation with the director general of the administration and the districts' chief engineers; they are then approved by the Ministry of Transportation. The results are monitored over the following year. As an example of such maintenance objectives, the 1990 light-pavement objectives have been as follows:

- The bearing capacity of oil gravel roads should not get any worse.
- The road condition (defects) index should be at least 3.5 on roads where traffic volume is above 800, and 2.5 for roads under 800.

Using the HIPS models, the optimal objectives, road policy, and strategy are defined for the long and short term, to bring the road network to desired condition levels. The districts then prepare the capital program, including the definition of

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projects, actions, locations, and costs. Accomplishment of the objectives is judged by annual measurements taken in the fall.

The actual implementation of maintenance measures is done either by the districts' own forces or by contractors. The central administration sets maintenance and design standards for this work and provides funding consistent with these standards.

To address this division of responsibilities, the pavement management system consists of two separate software packages:

- Network level, embodied in HIPS, which addresses abstract categories of roads and the allocation of funding among them. HIPS helps the central administration determine the funding level for each district and develops policy guidelines on how the money is to be used.
- Project level, embodied in PAS, which guides the district engineer in matching the network-level policies with detailed data bases of roads that already exist in each district.

Although both the districts and the central administration are potential users of the entire system, the central office is the primary user of the relatively abstract modeling and allocation procedures in HIPS, whereas the districts are the primary users of the more detailed PAS. Several levels of analysis are provided in HIPS to address the capital programming policy questions of interest to the highway administration. These are as follows:

- Pavement type level distinguishes asphalt concrete pavements from light pavements.
- Region level distinguishes the cold and relatively dry interior and northern parts of the country from the more temperate and moist southern coastal area. Climatic differences affect the behavior of pavement deterioration and the choice of maintenance policy.
- Volume class level affects the rate of deterioration of pavements as well as the level of user costs associated with pavement condition. Low-volume, oil gravel roads are those with average daily traffic below 350, high-volume roads are those above 800, and medium-volume roads are those in between. As traffic volume on a road increases above 800 toward 1,500, it receives consideration for paving with asphalt concrete. Much of the modeling work occurs on the level of the 12 permutations of pavement, region, and volume class, termed the "P/R/VC" level.
- National level accumulates the results from the pavement types, regions, and volume classes to address national funding levels and funding allocations.
- District level reflects the 13 maintenance districts that are the recipients of capital funding and that carry out all of the programmed work. Each district is contained in one region and has roads representing all three volume classes.

Table 1 summarizes the light-pavement network at the region/volume class level.

As in any far-sighted capital programming process, the roads administration is concerned with the long-range goals that should be established for the highway network, and also with the steps needed to proceed from the current situation toward the long-range goals. Because the projects addressed by the pavement management system are largely ones of in-kind facility replacement and major maintenance, the roads administration would like decision making in this area to concentrate on direct economic benefits and costs to road users. This expectation leads to the next important division within HIPS:

- The long-term model analyzes possible long-term goals and tries to find a future policy that minimizes social costs (the sum of user and agency costs) and is sustainable indefinitely into the future. The long-term model is not tied to the current condition of the network and imposes no requirements on which specific year it should be achieved.
- The short-term model's first priority is to find the quickest practical means of achieving the level of network condition that would make the long-term policy possible, and its second priority is to minimize the social cost incurred in the short-term period between now and the time when the long-term goals are achieved.

As shown in Figures 1 and 2, the flow of activities in using HIPS starts at the most abstract level and ends at the most concrete level. The long-term model analyzes the general behavior and cost structure of roads in each of the six subnetworks of region and volume class, and then uses an incremental benefit-cost model to determine the best allocation of funding among them. It defines goals broadly and at some undetermined time in the future. This then proceeds to the short-term model (Figure 2), which is more concrete because it is explicitly tied to the current observed condition of the road network (from the Road Data Bank). Following this, the analysis becomes even more concrete in the short-term budget allocation steps, an activity of immediate interest to the managers within the administration and the districts. Finally, the least abstract activity is the definition of actual projects on specific roads, in PAS. This flow of abstraction follows the general flow of the administration's planning process, and provides a way in which the economic merits and costs of rehabilitation policy can be conveniently merged with the noneconomic and political considerations that also determine the ultimate budget allocations and capital program.

TABLE 1 LIGHT-PAVEMENT CHARACTERISTICS IN FINLAND

Average Daily Traffic (vpd)	Road Length (km)		Pavement Width (m)		Average Speed (kph)	
	North	South	North	South	North	South
<350	15,363	7,424	5.7	6.0	45	45
350 to 800	12,309	7,828	6.3	6.3	62	45
>800	5,258	3,419	6.6	6.6	67	55

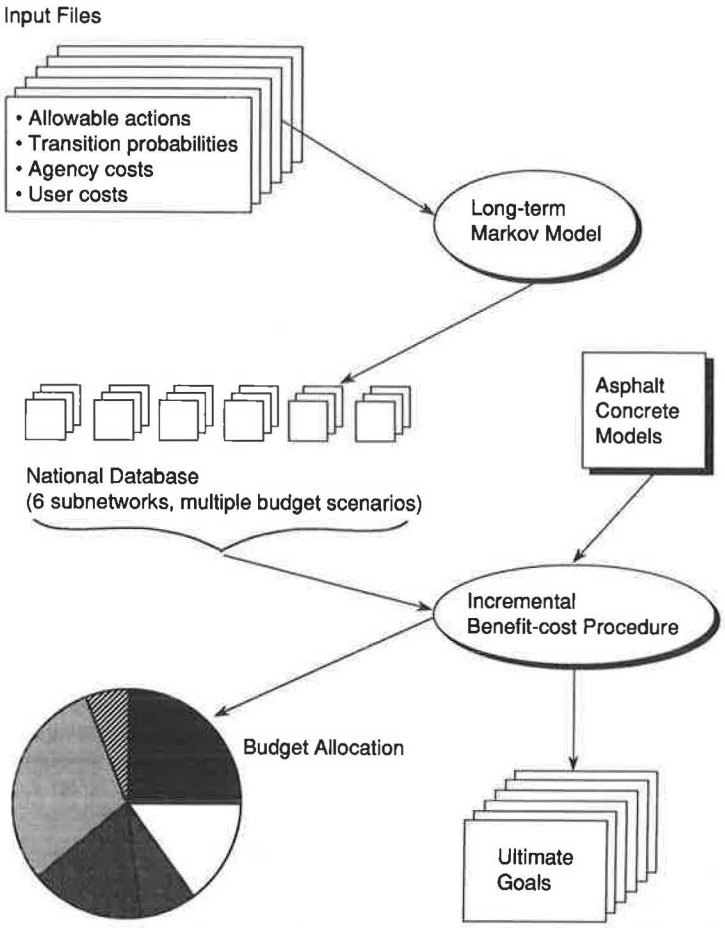

FIGURE 1 Long-term goal setting.

In both the long-term and short-term analyses, the central feature is an economic model of pavement behavior, rehabilitation policy, and their combined social cost implications. The main components of the economic model are as follows:

- Agency cost model, giving the average construction costs for five general categories of rehabilitation action, from do nothing to total reconstruction.
- User cost model, which quantifies in economic terms the increase in travel time, fuel consumption, and vehicle wear-and-tear associated with deteriorated road condition. Even though the absolute level of user costs on light pavements is quite low, because of low traffic volumes, it is expected that the high dependence of travel speeds on rehabilitation policy will still have a significant effect on the selection and timing of actions.
- Deterioration model, describing the process by which a road deteriorates and thereby causes higher user costs to be incurred. Similarly, it also describes the improvements that can be expected after each of the general rehabilitation actions is applied.

As the expenditure of agency costs increases, the resulting level of user costs is expected to decline, as long as the available money is always used in the most cost-effective manner; also, as agency costs decrease, user costs go up. The economic optimization framework assumes that there is an intermediate point at which social cost, the sum of user and agency costs, is minimized. Policy questions that are addressed in the framework are as follows:

- What is the optimal (minimum social cost) level of expenditure on oil gravel pavement rehabilitation and on major maintenance on the nationwide road network, and what are the optimal expenditures within selected subnetworks?
- At funding levels that do not minimize social costs, what is the optimal allocation of funding among subnetworks, and what is the most cost-effective means of spending the available money? What is the best overall allocation among action types, and what specific actions should be applied to the various kinds of roads?
- To what extent do budget constraints increase the level of costs borne by road users, and what does this imply about the importance to society of user costs relative to agency costs?
- How much better is the long-term optimal solution than the current situation, and how long must it take to achieve the long-term goals?

It should be emphasized that the issue of accessibility, which is the major justification given for the construction of oil gravel roads, is not a factor in the choice of actions on existing

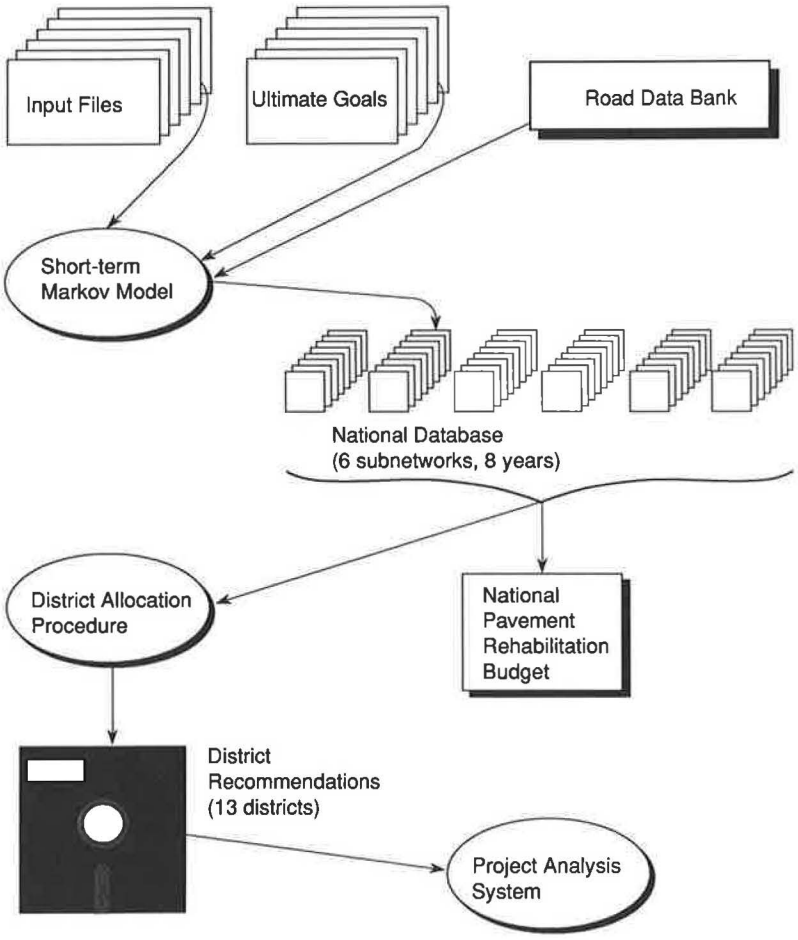

FIGURE 2 Short-term budgeting.

light surfaces. It is assumed in the models that existing oil gravel roads will be kept at least passable to all traffic.

Many different modeling methodologies can be applied to these questions. At the beginning of the earlier development effort for asphalt concrete, a thorough review was conducted of the experiences of U.S. and international organizations using many different techniques. The methodology finally selected was an adaptation of Markov dynamic programming, a technique that had been used in optimization applications as diverse as fleet replacement, catalog mailing list selection, timing of bond calls, and purchase of satellites (*3*). At the time, the only full-scale implementation of the technique in pavement management was in Arizona (*4*). Attributes of the Markov model that made it attractive were as follows:

- It is intuitively appealing to top managers in the administration, as it describes the behavior of pavements in a manner that is simple and fits well the organization's structure and decision-making processes. Also, it is an innovative approach that can be useful to other countries if it works out well.
- It explicitly recognizes the stochastic nature of pavement behavior, and therefore expresses its conclusions in a form that, in comparison to deterministic models, can more readily be defended against anecdotal evidence. This feature is especially important when introducing the technique to skeptical district engineers.
- Creative and relatively simple techniques can be used to aggregate or allocate the conclusions of the models to higher and lower levels of abstraction for different purposes.
- It can be implemented on a microcomputer.

The point concerning microcomputers was not immediately obvious for a road network of the size in Arizona when the model must be solved by formulating it as a linear program using specialized computationally intensive software. However, the key to a successful microcomputer solution is the separation of the abstract, network-level model from the detailed data base of potential actual projects, so that the computational requirements of the two do not compound each other. The network-level model optimizes a small number of general categories of roads, defined by their geographic region, volume class, and condition classes. The definition of categories must be designed carefully to give the needed policy sensitivity, but they operate at a level of aggregation with which the central administration is most comfortable. After the optimization is completed, the resulting recommendations are transmitted on a floppy disk to the district offices, where they are applied to the large detailed data base of actual road segments using less computationally intensive techniques in PAS.

Both HIPS and PAS are menu oriented and user friendly. All through the system, analytical and summary reports and screen displays are available to show how the models are developing. A convenient filing system manages the input and output files on the hard disk and allows the user to keep track of multiple versions of an analysis archived on floppy disks or hard disk subdirectories. All activities involved in using HIPS are available from a menu hierarchy; at each leaf of the menu tree, a screen representing the module or report explains the purpose of the module and allows the user to fill in various options that are available for that module.

INPUT DATA AND ORGANIZATION OF THE MARKOV MODELS

The formulation and organization of the Markov models is loosely based on the formulation used in Arizona, but conditions are categorized in a much different way, and user costs are included explicitly in the objective function. Also, the desire to implement the system on microcomputers has introduced both design constraints and design opportunities that, in the end, have led to a system that is unique. Markov dynamic programming can be distinguished from other optimization approaches by several features:

- Problems are structured into multiple stages, which are solved one stage at a time. All the stages are structured identically to each other, and all have the same possible outcomes. The stages are evenly separated from each other, usually in time, in a uniform progression.
- The range of possible outcomes of each stage is expressed as a set of discrete states. It must be possible to write out a reasonably short list containing every possible result.
- The outcome of any particular stage depends stochastically on the outcome of the stage before it, and not on the outcomes of any of the other stages. Because of this, Markov dynamic programs are said to have "one-step memories."

By applying a Markov model recursively over a series of stages, it is possible to predict probabilistically the outcome of any future stage. Such a series of Markov predictions is called a Markov chain.

For the purposes of Finland's pavement management system, each stage is a description of the condition of the road network in a given year in terms of the distribution of roads among the set of possible states, combined with the choice of action taken in that year. Figure 3 shows how, in a system of five states, a Markov chain of deterioration plays out for a road starting in the highest state. As expected, the road ends up in the lowest possible state eventually, but the path it uses to get there may vary. Although such a stochastic prediction may be of limited use in designing the treatment for any particular road, it is useful for characterizing a whole road network.

Pavement deterioration and resulting capital needs are driven by the effects of climate and traffic, which cause both visible damage to the pavement and hidden loss of resistance to further deterioration. The visible damage—roughness and rutting—increases user costs and makes it necessary to invest in maintenance. Measurements of all of these factors must exist in any complete pavement management system. HIPS makes an important separation between factors that are exogenous and factors that are affected by maintenance policy. The endogenous factors measured in Finland are

- Longitudinal and transverse roughness, which most directly affect user costs;
- Distress index, a weighted average of several distress measures, which is visible evidence of pavement weakening and damage that may lead to future roughness; and

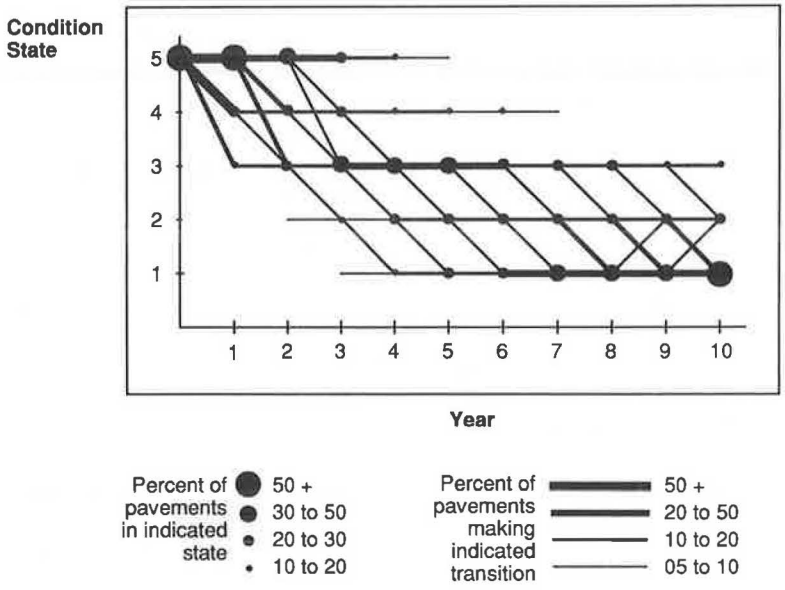

FIGURE 3 Markovian deterioration.

- Bearing capacity, which at lower values presages future distress and roughness.

Roughness is measured by a bump integrator (longitudinally) and a rod and ruler (transversely); distress is estimated visually. Finland is unusual in the large extent to which it measures bearing capacity, driven by the need to monitor the damage of annual freeze-thaw cycles. In countries that do not experience such damage, monitoring of bearing capacity is far less important.

To set up the optimization data base, Finland's oil gravel roads are surveyed for these four condition variables, and then placed in discrete categories: three categories each for longitudinal roughness, transverse roughness, and distress; and four categories for bearing capacity. The combination of these categories yields the 108 condition states.

Climate and traffic, the exogenous factors, are incorporated in the models by classifying all roads into two regions and three volume classes. Each of the six combinations of these variables is modeled separately; a set of optimal investment policies, with resulting user costs, is generated at several alternative budget levels for each one. An incremental benefit-cost procedure uses the alternative budget levels to choose the most equitable national allocation of funding. Traffic growth is modeled by the projected change in kilometers in each traffic volume class.

One strength of the Markovian methodology is that traffic growth and network size have no impact on the size or complexity of the optimization models; they work just as well on large networks or small ones, and policy recommendations within each traffic volume class are not affected by growth. Only after the optimal policies are determined is it necessary to consider the impact on individual roads, and this separation between optimization and data base manipulation is what makes the process simple and implementable on microcomputers.

Data have been developed by the administration to estimate the deterioration and cost models. For the do nothing action category (which includes routine maintenance), transition probabilities are directly estimated from the Road Data Bank; for other actions, probabilities are developed from a Delphi process of interviewing expert engineers. Because of the memory-less property of Markov models, only 2 years of data on a sample of roads is required for model estimation, and the only data items required in Finland's models are traffic volume, roughness, defects, and bearing capacity.

Model estimation is a simple matter of observing the roads in each condition state in the first year, and observing the fraction of them that transition to each other state in the second year. Monitoring, validation, and updating of the models is equally easy and is a necessary part of routine pavement management. Agency cost factors are developed from the administration's own historical records, whereas user costs are estimated by the administration's research center from experimental evidence and economic data resulting from more than 10 years of research (*5*).

LONG-TERM MARKOV MODEL

Mathematically, the long-term Markov model assumes a steady state distribution of pavements among the 108 condition states. This does not mean that each road is always in the same condition, but it does mean that, in every year, the same overall fraction of roads may be found in the same state. It also means that the same fraction of roads undergoes the same general action each year. This is all part of the requirement that the long-term program be not only optimal, but also sustainable, indefinitely.

Of course, the condition distribution among the 108 condition states measured today is generally not equal to the long-term optimal distribution. In fact, the long-term model is not in any way tied to the current condition distribution or current rehabilitation policy, but represents instead a goal that might be attained at some time in the future. What makes the goal desirable is that it minimizes social costs. For the purposes of translating the model to a linear program, the social cost of the long-term program is calculated in the following manner as the objective function to be minimized:

$$\text{Social Cost} = \sum_a \sum_i W_{ai}(C_{ai} + \phi U_{ai}) \tag{1}$$

where

W_{ai} = decision variable, the fraction of all pavements that are in State i and have Action a applied to them (there are 108 states and 8 actions);
U_{ai} = user cost factor in marks per kilometer;
C_{ai} = agency cost factor in marks per kilometer; and
ϕ = degree of user cost contribution to the objective function, usually 1. (It can be varied in an automated parametric analysis provided in HIPS.)

To prevent leakage from the system, and to give scale to the W_{ai} decision variables, the first constraint on the linear program is a definitional unity constraint:

$$\sum_a \sum_i W_{ai} = 1 \tag{2}$$

The most important element of the formulation is a constraint that combines the Markov model with the requirement of a steady state:

$$\sum_a \sum_i W_{ai}P_{aij} = \sum_a W_{aj} \qquad \text{for all } j \tag{3}$$

where P_{aij} is the transition probability of going to State j in Year $t + 1$ (given State i in Year t, when Action a is applied in Year t), which does not depend on t.

This constraint starts with the distribution of condition states and actions in Year i, applies the transition probabilities to get the distribution in the next year, Year j, and then says that this resulting distribution must be the same as in the previous year, so that all years have the same condition distribution. Loosely interpreted, this means that, for every kilometer of road leaving any given state, another kilometer must arrive in that state to replace it. Next comes an optional budget constraint, which can force the agency cost total to a level either higher or lower than the social-cost minimization level.

$$\text{BMIN} \le \sum_a \sum_i W_{ai}C_{ai} \le \text{BMAX} \tag{4}$$

where BMAX and BMIN are budget constraints (marks/km) that can be varied in an automated parametric analysis.

Finally, the long-term model has optional condition constraints. To make these most relevant and usable, condition constraints are applied to condition classes, rather than states. There are 4 bearing capacity classes, 3 defects classes, 3 rutting classes, and 3 roughness classes, for a total of 13. Each state belongs to four different classes, one in each condition variable. The constraints are

$$\text{CMIN}_c \leq \sum_{i \in c} \sum_{a} W_{ai} \leq \text{CMAX}_c \qquad \text{for all } c \tag{5}$$

where CMAX_c and CMIN_c are fractions that represent the limits on the total fraction of pavements allowed to be in each class c.

The parametric analyses on user cost contribution and budget constraints are important. Both capabilities produce a series of different scenarios in which different levels of agency costs are incurred and different levels of user costs result. These scenarios are used in the allocation of resources among the subnetworks, as described in the next section.

ALLOCATION AMONG REGIONS AND VOLUME CLASSES

By setting up and running a long-term Markov model for each permutation of pavement, region, and volume class, the analyst can determine the social cost–minimizing pavement rehabilitation policy for each subnetwork. But the agency cost required to maintain this policy may not be within the range of funding that the administration can expect. To make the model more realistic, budget constraints are required. The question is, what constraint should be applied to each subnetwork? or, put another way, how should funding be allocated among them?

A way of looking at this issue is to assume that the national funding level for pavement rehabilitation represents the relative value that society places on user costs as opposed to agency costs. In HIPS, this value can be analyzed by varying the factor ϕ in the long-term objective function. One way to conduct budget allocation is to use the same ϕ in all 6 models (12 models when asphalt concrete roads are included) and to experiment with the value of ϕ until a set of solutions is found whose agency cost equals the expected national funding level.

HIPS conducts an analysis similar to this in a systematic manner by using a variation of the incremental benefit-cost technique. As Figure 4 shows, a long-term parametric analysis (either on the user cost contribution ϕ or on the budget constraints) produces a user cost versus agency cost curve that is always concave upward. Between any two parametric steps, the quotient of change in user cost divided by change in agency cost is called the "shadow price." The shadow price is always negative and always becomes less negative as agency cost increases.

A criterion for an economically efficient allocation of resources among the 6 or 12 subnetworks, at funding levels that do not minimize social costs, is that the shadow price is equalized among the subnetworks. In an allocation where this is true, it is not possible to save additional user costs by shifting money from any one P/R/VC to any other. If the user executes a parametric analysis for all 6 or 12 models, then the incremental benefit-cost procedure can use the algorithm shown in Figure 5 to create sets of allocations at varying budget levels that roughly equalize the shadow prices. In practice, the user first sets up a broad parametric analysis for each P/R/VC, including the highest and lowest conceivable budget levels, to give a rough allocation of funding over a wide range of budget scenarios. Then the parametric analyses are conducted again for the smaller budget ranges suggested by the first report, and the allocation report is reprinted to give more precise recommendations.

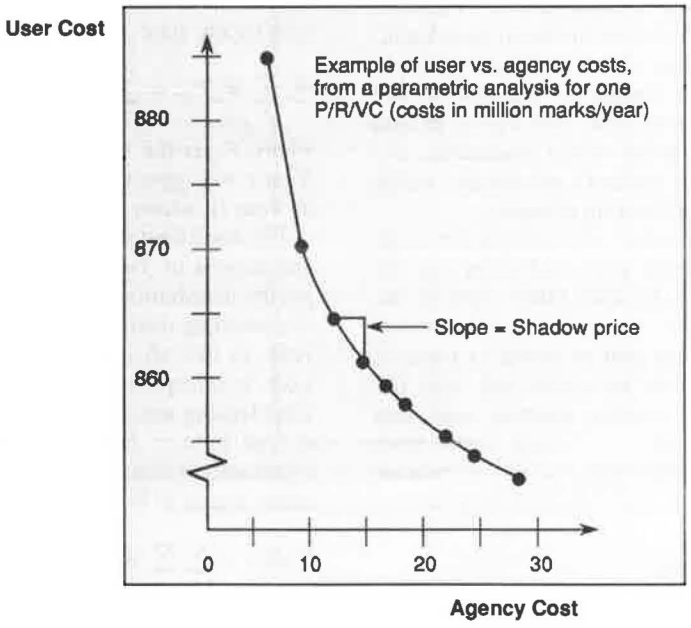

FIGURE 4 Long-term parametric analysis.

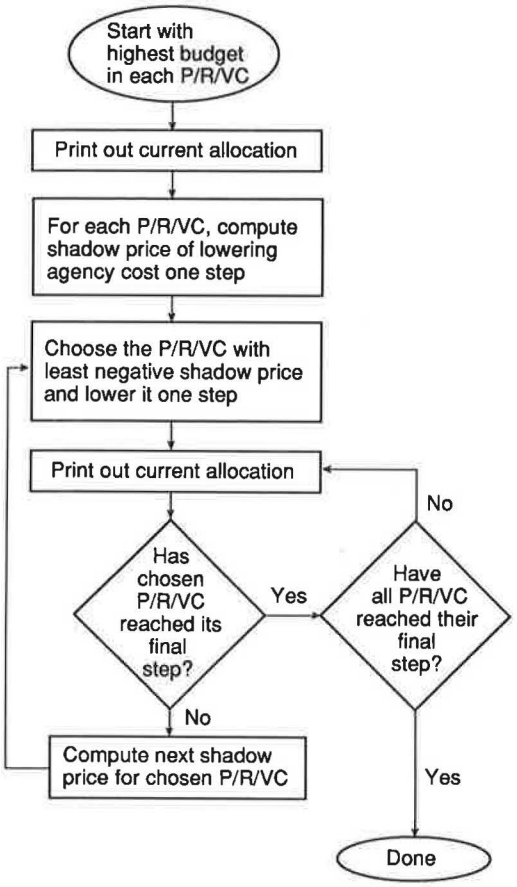

FIGURE 5 Incremental benefit-cost allocation.

SHORT-TERM MARKOV MODEL

All of the analysis described so far focuses on an optimal scenario that may take place at an undetermined time in the future. What is needed next is a capability to model the steps required to bring the existing road network to the condition level needed by the chosen long-term solution. The purpose of the short-term model is to find the shortest practical means of achieving the long-term condition distribution, minimizing as much as possible the social costs incurred along the way.

This is done by modeling each year separately in the short term, with the objective function trying to minimize the deviation between the condition distribution at the end of the year and the long-term optimal distribution. In other words, the short-term model tries to maximize the amount of progress made each year. Mathematically, the objective function is

$$\text{Min} \sum_c (K_c X_c + K_c Y_c) \tag{6}$$

where

X_c = amount by which the fraction of pavements in Class c at the end of the year is above the long-term optimal fraction (0 if it is below the long-term fraction);
Y_c = amount by which the fraction of pavements in Class c at the end of the year is below the long-term optimal fraction (0 of it is above the long-term fraction); and
K_c = average social cost (marks/km) of Class c. K_c is computed from the long-term steady state condition distribution and the unit agency and user costs by the expression

$$K_c = \left[\sum_{i \in c} \sum_a W_{Tai}(C_{ai} + U_{ai})\right] \Big/ \left[\sum_{i \in c} \sum_a W_{Tai}\right] \tag{7}$$

where

W_{Tai} = long-term fraction of pavements in State i with Action a,
U_{ai} = user cost factor (marks/km), and
C_{ai} = agency cost factor (marks/km).

The weights K_c placed on the objective function variables ensure that the classes having the highest social costs receive the highest priority. Using the X_c and Y_c variables allows the Markov constraints to be expressed in terms of the long-term condition distribution, as follows:

$$\sum_{j \in c} \sum_a \sum_i W_{ai} P_{aij} - X_c + Y_c = \varepsilon_c \qquad \text{for all } c \tag{8}$$

where

P_{aij} = transition probability of going to State j in Year $t + 1$, given State i in Year t, when Action a is applied in Year t; and
ε_c = short-term ultimate distribution among condition classes, usually computed as a summation of the long-term state distribution in the same manner as the long-term condition constraints.

Because this formulation is run separately for each year, a constraint is needed for the initial condition distribution at the beginning of the year. In the first year, this constraint is the known distribution that has been measured in the field and is maintained in the Road Data Bank. In the subsequent years, the constraint is the distribution output from the preceding year's solution.

$$\sum_a W_{ai} = q_i \qquad \text{for all } i \tag{9}$$

where q_i is the condition distribution at the beginning of the year.

The short-term model also has optional budget constraints, which are identical in structure to those used in the long-term model. A report is provided in HIPS to allocate a fixed short-term budget level, above and beyond the amount needed to maintain the long-term distribution, according to the amount of improvement needed to reach the long-term solution; the amount of improvement needed is the Year 0 value of the short-term objective function. Normally, the analyst's judgment and noneconomic considerations also play a major role in setting the short-term budget levels.

A file of ultimate goals for the short term is prepared from the long-term solution, expressed as the fraction of pavements in each of the 13 condition classes. The user is free to edit these goals and the weights associated with each condition class. To run the short-term model, the user gives a series of maximum and minimum budget levels for each year, and then

starts the optimization process. HIPS automatically executes a sequence of eight optimizations, one for each year, and places all of the results in a short-term results file for later reporting. Over the short-term period, the condition distribution approaches the long-term optimal distribution asymptotically, as long as the short-term budget level remains above the long-term agency cost.

An alternative short-term approach, in principle, is to add to the long-term formulation a state variable representing time. For the administration's 8-year planning period, then, there would be 108 condition states multiplied by 5 actions, multiplied by 8 years, for a total of 4,320 states. The model could plot a series of yearly policies in a Markov chain, culminating in a constraint at the end of the eighth year that would force the model to reach the optimal long-term distribution. A short-term model of this sort, for a period of 5 years, has been developed and solved in Arizona's system on a mainframe computer. Although the approach chosen for HIPS does not have the technical guarantee of optimality that the alternative would, it has shown in 2 years of use with asphalt pavements that it produces practical recommendations that are consistent with the long-term optimum, and it has the distinct advantage of being operable on a microcomputer.

IMPLEMENTATION AND PERFORMANCE ISSUES

A concern frequently voiced with Markov models for pavement management is the perception that the models require too much data and computing resources. The HIPS development team shared this concern at the outset of its asphalt concrete system in 1985, but has since proven that the requirements do not have to be so onerous.

The nature of the five data items collected for Finland's roads to support HIPS (longitudinal and transverse roughness, defects, bearing capacity, and traffic volume) is by no means unusual (*6*). Measurement of roughness, defects, and traffic are standard for most pavement management systems; bearing capacity is uncommon, but can be replaced by other more easily obtained information such as Arizona's index to first crack and change in cracking (*4*) if bearing capacity is unavailable or (for warmer countries) irrelevant. Estimation of deterioration models requires a smaller time series (2 years) than any other model form (*7*). User cost factors are the most difficult to estimate, but the administration's research (*5*) has given reliable data to supplement earlier work in Kenya (*8*), Brazil (*9*), the United States (*10*), and the original Winfrey tables (*11*).

Performance of the models on personal computers has also been better than expected. On a 33-MHz IBM-compatible computer, solution times for the 1989 long-term asphalt concrete models was 10 to 20 min; the short-term models took about half as long. With only 108 states (as opposed to 135) and 5 possible actions (as opposed to 8), the light-pavement models are expected to have solution times one-third to one-half as long as the asphalt models. Because the costs of developing and modifying software on personal computers are far less than on other platforms (*12*), there has been no need to consider using a more powerful machine.

CONCLUSIONS

The asphalt concrete models in HIPS have been in use in the roads administration since the summer of 1988, and the oil gravel models are scheduled to be completed in the fall of 1990. The asphalt models have proven to be stable and to give reasonable and useful recommendations, which have been heavily relied on in the administration's budgeting process. Managerial support of the system and confidence in its recommendations continues to be high.

The Finnish experience with Markov models in pavement management shows that they can be implemented on a microcomputer, in a form that is fast, user friendly, and understandable. Their most attractive feature is the stochastic nature of their performance models and recommendations. Perceived drawbacks to the approach, such as the computational intensity of solution procedures, can be overcome. A continuing challenge now facing the administration is to continue to improve the models as new data are collected, and to monitor the implementation of the recommended rehabilitation programs to see if the long-term condition objectives are achieved.

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Recent Policy and Legislative Actions To Pave All Unpaved Secondary Roads in North Carolina

DAVID C. ROBINSON AND THOMAS R. KENDIG

North Carolina has the largest state-maintained highway system in the nation, consisting of 76,727 mi. Of this total, 59,310 mi (77 percent) is on the rural state secondary system. As of January 1989, 16,358 mi (28 percent) of the rural state secondary system remained unpaved. Between 1981 and 1988, the department was only able to pave 2,293 mi of unpaved rural state secondary roads, an average of 287 mi per year. At this rate, it would have taken 55 years to pave all the remaining unpaved rural secondary roads in the state. Faced with the inadequacy of the existing revenue source—a motor fuel tax of 1¾ cents/gal generating about $68 million annually—for all types of secondary road paving, the 1989 North Carolina General Assembly created the North Carolina Highway Trust Fund, which is designed to provide $9.2 billion of additional funding for new primary and secondary highway and bridge construction throughout the state over a 13½-year period. The policy decisions and legislative actions behind North Carolina's secondary road paving program and the manner in which candidate projects are ordered by priority are explored. The discussion should be of interest to all state and county highway agencies responsible for low-volume road paving and maintenance.

North Carolina's secondary road system evolved from a limited network of county roads that the state helped the counties plan and design in the years during and immediately following World War I. In 1921, public demands for better roads resulted in the state's taking over 5,200 mi of county roads and levying a 1-cent/gal fuel tax to maintain them.

During the period 1921 through 1927, North Carolina became known as "The Good Roads State," when the approval of $115 million in bonds financed the construction of a system of hard-surfaced roads connecting each of the 100 county seats. The nationwide depression of 1931 rendered the counties financially unable to maintain their roads, so the general assembly voted to assume responsibility for all county roads, with the exception of city streets. This added approximately 47,000 mi to the state highway system, bringing the total primary and secondary mileage to 57,000 by 1934.

In 1934, the motor fuel tax was at the 6-cents/gal level, where it would stay until 1949, when it was raised by 1 cent to pay debt service on $200 million of exclusive secondary road bonds. At that time, however, there was no dedicated revenue for secondary road paving from the motor fuel tax.

As a result of actions by (a) the 1951 general assembly's authorizing the state to take over those city streets that were part of the state highway system and dedicating ½ cent of the motor fuel tax to be given to the cities for off-system street improvements, and (b) the 1971 general assembly's raising the dedicated amount from ½ to 1 cent/gal, the annual appropriation for secondary road paving was finally tied to the motor fuel tax by statute, being set at 1 cent/gal, the same as the appropriation for the cities. By 1971, the state highway system comprised just over 74,000 mi (of which 59,000 mi was rural secondary roads), and the motor fuel tax was 9 cents/gal.

In 1981, the amount of the motor fuel tax dedicated to secondary road paving was increased to 1⅜ cents; this was further raised to 1¾ cents in 1986. Because 1 cent of motor fuel tax collected in North Carolina generates about $40 million in revenues, the secondary road paving program received an allocation of $68 million in 1988.

The initial dedication of 1 cent/gal for secondary road paving in 1971 was followed by legislative action in 1973 to establish a fair and objective method of determining paving priorities and allocating funds to geographic areas (that is, counties).

Figure 1 shows how the paving program has been able to reduce the number of unpaved miles of secondary roads from over 45,000 in 1935 to just under 16,000 in 1990. The slight annual increase in the combined total of unpaved and paved miles reflects the inclusion of paved rural subdivision streets that meet state design standards.

RATING UNPAVED SECONDARY ROADS

North Carolina General Statute (NCGS) 136–44.5 requires the Department of Transportation to make a study each calendar year to determine the number of miles of unpaved state-maintained roads in each county and in the state. The amount of secondary road paving funds (C_i) allocated to County i is calculated as the product of the statewide secondary road construction appropriation (S) and the ratio of the county's unpaved secondary road mileage (M_{ci}) and the statewide total unpaved secondary road mileage (M_s).

$$C_i = S \times \frac{M_{ci}}{M_s} \quad (1)$$

NCGS 136–44.7 further stipulates that "Projects on the annual construction program for each county shall be rated according to their priority based upon the secondary road criteria and standards which shall be uniform throughout the state. . . ."

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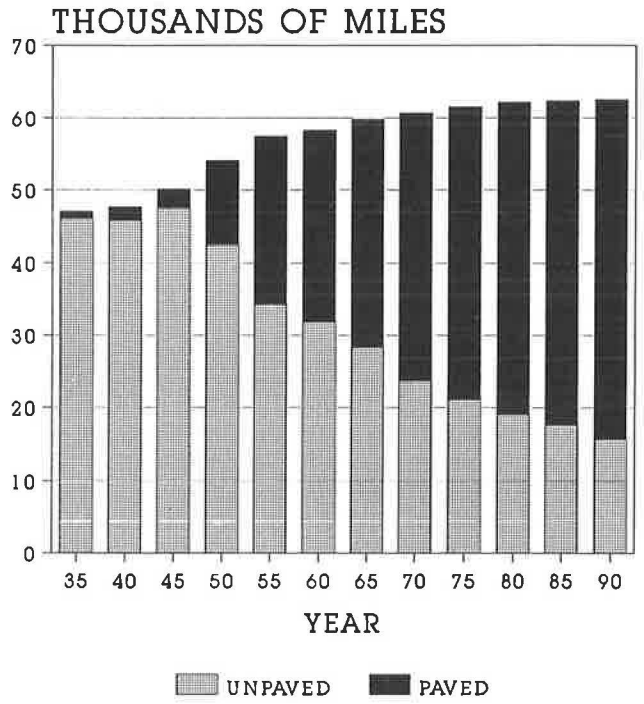

FIGURE 1 Secondary road mileage.

A rating is obtained for each unpaved secondary road in each county by summing the scores obtained from evaluating the following land use, traffic, and general route characteristics:

Land Use (total score divided by mileage)

- Homes (6 points/year-round home, 3 points/seasonal home);
- Schools (10 points/school);
- Churches (10 points/church);
- Businesses (5 points/business);
- Industries (5 points/industry); and
- Recreational facilities (5 points/public or nonprofit).

Traffic

- School bus route (10 points); and
- Average annual daily traffic volume (value).

General Route

- Value of road as a county thoroughfare route (10 points maximum).

A priority list is developed for each county, and paving is scheduled according to the order of projects on the list.

This system worked well during the 1970s and early 1980s. However, by the mid-1980s, the department recognized that the success of the allocation program had created inequalities. For instance, in a number of counties, the unpaved mileage had been reduced to a level that the funds allocated to those counties were insufficient to cover the cost of finishing the paving job.

Also, in some cases, counties with low population densities or low construction costs, or both, have had more miles paved per unit expenditure than other counties with high population densities or high construction costs, or both. The result is that relatively low-volume secondary roads have been paved in some counties, whereas relatively high-volume secondary roads remain unpaved in other counties.

To compound these inequalities, the expansion of suburban and rural residential development along secondary roads has caused traffic congestion and safety problems for which widening offers the only solution. The recognition of these inequalities prompted the 1987 general assembly to include secondary road construction needs along with primary system construction needs when it created the joint legislative highway study commission and asked for recommendations to be made to the 1989 general assembly.

HIGHWAY STUDY COMMISSION

The highway study commission was created by the 1987 general assembly for the purpose of

1. Reviewing and validating present and future highway funding needs,
2. Evaluating the economic impact of the highway system on the state and its various regions,
3. Reviewing and developing recommendations to ensure that the highway funds are allocated to meet the identified needs and address the overall growth and economic goals of the state, and
4. Recommending both short-range and long-range funding solutions with particular emphasis on the separation of general fund and highway fund revenue bases and sources.

The study commission was composed of 15 members, including 5 members appointed by the president of the senate, five appointed by the speaker of the house, and five appointed by the governor. The appointees—nine were legislators, three were local elected officials, and three were from the private business sector—represented 30 rural counties and 7 urbanized counties out of a statewide total of 100 counties.

The study commission held its first meeting in November 1987, and its 20th and last meeting in February 1989. During this 16-month period, the Department of Transportation was afforded ample opportunity to brief the commission on the construction and maintenance needs both of primary and secondary highways. In addition, the commission heard from other state agencies, highway departments in other states, and a number of interest groups representing municipal and county governments, consulting engineering and construction firms, as well as the general public (at four special meetings held out of Raleigh).

Early in the commission's deliberations, the Department of Transportation provided the commission with a statement of highway needs, which revealed unmet needs of $12 billion for primary highway construction and $1 billion for maintenance through 2000. The secondary road program was shown to have backlog needs of almost $1.5 billion, with additional needs through 2000 amounting to about $1.5 billion, of which only $0.75 billion could be expected to be accomplished under

the existing program. This amount left a total of $2.25 billion in secondary road paving needs through 2000—and still there would be unpaved secondary roads in North Carolina.

Faced with this uncomfortable prospect, and the political expediency to satisfy both rural and urban interests, the commission asked the Department of Transportation to provide its members with two stratifications of unpaved secondary road mileage: those above and below 100 vehicles per day (vpd), and those above and below 50 vpd.

Out of a total of 16,932 unpaved secondary road miles, it was determined that 3,534 (20 percent) were carrying in excess of an average of 100 vpd. When the criterion was dropped to 50 vpd, the number of miles exceeding it was found to be 9,514 (56 percent). When the commission learned that using the 100-vpd criterion for establishing a priority paving program would mean that some counties would be excluded from such a program because none of their unpaved secondary roads carried 100 vpd or more, the commission chose the 50-vpd criterion. This choice would result in all counties sharing (to different degrees, admittedly) in any priority paving program that would be subsequently enacted into law.

The final recommendations of the commission were fourfold: (a) to complete 1,830 mi of multilane construction on the designated 3,600-mi North Carolina Intrastate System; (b) to construct multilane connectors or loops in 7 of the state's 15 urbanized areas; (c) to pave all secondary roads carrying 50 vpd or more within 10 years, and the remaining miles within a subsequent 6-year period; and (d) to increase funds available for projects in the Transportation Improvement Program.

The cost of accomplishing these recommendations was estimated to be a minimum of $8.6 billion over a 12-year period, and a financing plan involving the creation of the North Carolina highway trust fund, with revenues generated by a fuel tax increase and a tax on motor vehicle title transfers, was recommended.

THE LEGISLATURE

Within days of the final meeting of the highway study commission, the senate co-chairman introduced a bill into the senate (S290) calling for the highway study commission's recommendations to be implemented by raising the motor fuel tax by 3 cents/gal to 17 cents/gal plus raising the wholesale motor fuel tax from 3 to 7 percent, and collecting a 2 percent fee on all motor vehicle title transfers. Of the total amount accumulated in the highway trust fund each year, 65.05 percent would be used to design and construct the North Carolina Intrastate System; 24.06 percent for urban loops; 6.5 percent to supplement the regular secondary road appropriations to each county; and 9.39 percent to supplement the Transportation Improvement Program. The 1¾ cents/gal distributed to municipalities from the highway fund would be replaced by an appropriation equivalent to 9 percent of net highway fund revenues.

Simultaneously, a bill (H399) was introduced into the house of representatives by the house co-chairman of the highway study commission. The two bills were essentially the same, with differences only existing in the amount and type of legislative oversight of the Department of Transportation implementation of the program. However, as both bills made their way through various committees in each chamber, the provisions diverged. Also, the issue of a shortfall in general fund revenues was introduced into the debate, with the result that the financing provisions of the ratified bill (H399) were very different from those in the introduced versions.

Five months after the initial introduction of the highway trust fund bill, a compromise came out of a house/senate conference committee and was ratified. The largest public works program ever to be undertaken by a single governmental agency of North Carolina had been given the green light.

The highway trust fund program receives the following revenues (shown in Figure 2):

1. A highway use tax on motor vehicle title transfers. The tax equals 3 percent of the motor vehicle retail price (less any trade-in value) for new vehicles, or of the book value for used vehicles. It replaced the 2 percent sales tax collected on motor vehicle sales, which went into the general fund;
2. 25 percent of all revenues generated by the 17 cent/gal and 7 percent wholesale motor fuel taxes (initially equivalent to 20.9 cents/gal);
3. Fee increases for title certificates and other motor vehicle registration and driver licensing services;
4. All funds not needed to repay highway bonds after 1995; and
5. All interest and income earned by the highway trust fund.

As a result of the general fund shortfall, the sum of $636 million in highway use tax revenues is scheduled to be transferred from the highway trust fund to the general fund in the FY 1989 to 1991 biennium, after which the annual amount transferred is reduced to $170 million, which is the equivalent of the 2 percent sales tax that was repealed.

Once received, the net revenues (after the general fund transfers have been made) are allocated in the following manner:

1. Intrastate system—61.95 percent;
2. Urban connector and loops—25.05 percent;
3. Secondary roads—6.5 percent; and
4. Aid to municipalities—6.5 percent.

Secondary roads also receive $15 of the $30 increase in the title transfer fee.

The new formula for annually allocating secondary road paving funds has two components. One involves the allocation of $68 million (a hold-harmless amount). The other involves the allocation of the 6.5 percent and one-half of the title transfer fee (the secondary roads supplement) portions of the trust funds.

Therefore, the new dollar allocation (C_i) for County i is calculated as follows:

$$C_i = \left(68{,}000{,}000 \times \frac{M_{ci}}{M_s}\right) + \left(\text{Supplement} \times \frac{M_{ci50}}{M_{s50}}\right) \quad (2)$$

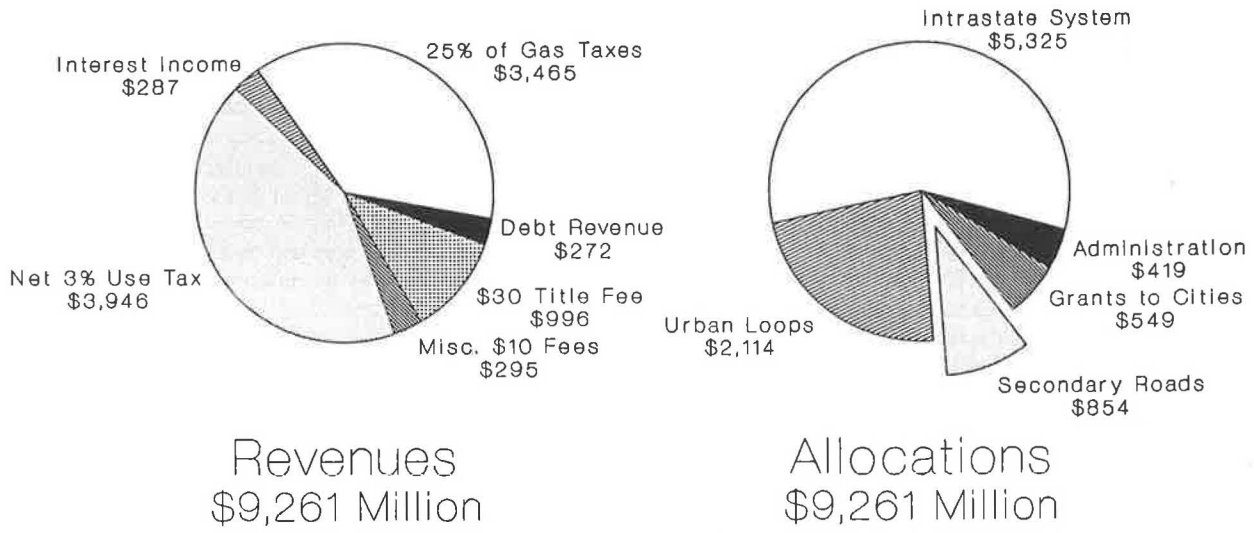

FIGURE 2 North Carolina highway trust fund revenues and allocations.

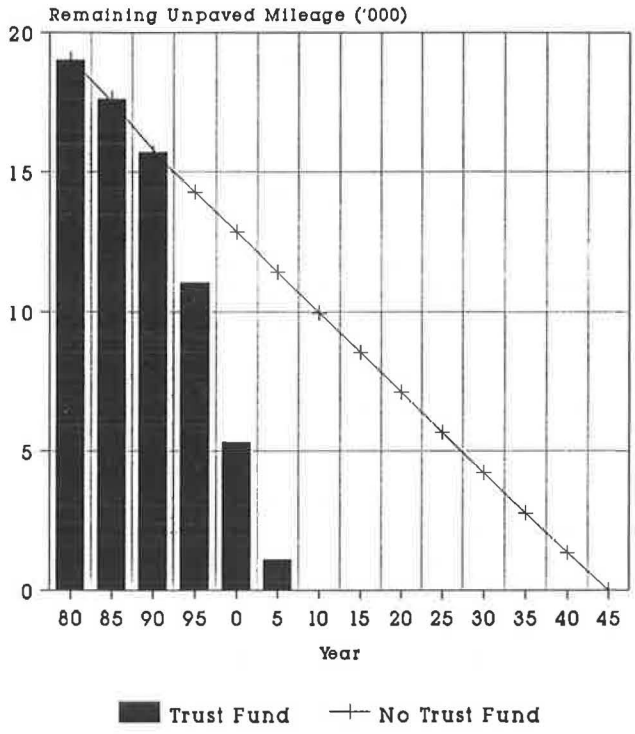

FIGURE 3 Unpaved secondary mileage comparison.

where

M_{ci} = unpaved SR miles in County i,
M_s = unpaved SR miles statewide,
M_{ci50} = unpaved SR miles, 50+ vpd, in County i, and
M_{s50} = unpaved SR miles, 50+ vpd, statewide.

In the unlikely, though possible, event that the 1¾ cents/gal motor fuel tax generates less than $68 million, then the supplement is reduced by the amount necessary to bring the coefficient of the first term in the equation up to $68 million. In the more likely event that the tax generates more than $68 million, then the excess is added to the supplement for allocation on the basis of the second term in the equation.

PAVING UNPAVED SECONDARY ROADS

The highway trust fund program is estimated to generate a secondary roads supplemental paving fund of over $850 million through FY 2000–2001, to which will be added approximately $840 million from the motor fuel tax revenue dedicated to secondary road improvements. This $1.69 billion expenditure will result in the paving of almost 10,000 mi of secondary roads with traffic volumes exceeding 50 vpd by FY 2000–2001.

An expenditure of $490 million from unsupplemented secondary road motor fuel revenues during the subsequent 6 years will complete the paving of all secondary roads with traffic volumes below 50 vpd. Unpaved roads in North Carolina will have been paved within the space of 17 years, as opposed to the 55 years it would otherwise have taken with existing funding sources.

Figure 3 shows the accelerated rate by which the unpaved mileage is reduced as a result of the trust fund program, and compares it with the rate that would have otherwise existed.

CONCLUSIONS

The paving of low-volume secondary roads in North Carolina has been highly political since the establishment of a system of state-maintained roads over 75 years ago. Over the years, the politicization of paving decisions has been reduced through the dedication, starting in 1971, of a specified amount of the

motor fuel tax specifically to be spent on paving secondary roads and, more significant, the introduction of the priority rating system and the allocation formula in 1973.

However, no allocation formula can be expected to remain equitable indefinitely, especially when the funds get distributed on the basis of remaining unpaved mileage, which is decreasing annually. So, the decision by the 1989 general assembly to create a supplemental paving fund to finally pave all remaining unpaved secondary roads within a specified time period, concentrating first on those with relatively higher volumes, was welcomed by both those whose responsibility it is to pave and maintain secondary roads and those citizens who live on unpaved roads. These citizens, before the new funding, had no expectation whatever of their road being paved in their lifetimes.

Now, these same citizens will soon be able to drive on paved roads from their homes to the new North Carolina Intrastate System, which is designed to reach to within 10 mi of 96 percent of all the citizens of North Carolina. The increased accessibility to employment, retail and commercial, and recreational opportunities created by the combination of a completely paved secondary road system and an extensive multilane primary road system will result in a stronger and more diversified economy for North Carolina.

Sub-Saharan Africa

Potential of Intermediate Means of Transport in Improving Rural Travel and Transport in Sub-Saharan Africa

JOHN D. N. RIVERSON AND STEVE CARAPETIS

To discuss the current situation, develop new policy directions, and specify needed and ongoing studies for improving rural travel and transport in sub-Saharan Africa, three hypotheses are postulated: (a) poor rural roads, with inadequate and high cost of road transport services and lack of affordable means of transport, seriously constrain rural economic and social development; (b) significant time savings and productivity gains can be achieved by using intermediate means of transport (IMTs) ranging from wheelbarrows to motorcycles, with significant impact on women; and (c) returns on investments in IMTs and low-cost rural infrastructure will be so high that private people will respond, with the government's role being largely promotional. Current rural travel and transport are dominated by head loading and walking (largely by women) to satisfy the daily travel and goods movement needs of rural populations in sub-Saharan Africa. Although rural roads and off-road transport may interact synergistically, with each amplifying the economic and social impact of the other, this interaction has not been directly studied or quantified. IMTs would save large amounts of time and energy and reduce drudgery, particularly for women. When IMTs have been introduced and used in the transport system, private individuals have usually developed and reaped the benefits. The better understanding that is required of the nature of rural travel and transport and its impact on rural development is being addressed in current studies under the sub-Saharan Africa Transport Program by the World Bank and the Economic Commission of Africa with the support of bilateral donors.

The gravity of rural transport bottlenecks in sub-Saharan Africa (SSA), has recently been underscored by the World Bank's long-term perspective study (LTPS) (*1*). This report emphasizes that improving rural infrastructure is an essential requirement for the modernization and growth of agriculture. Further, it cautions that although better market incentives (especially related to prices and inputs) to farmers remain an important factor in agriculture, the effects of these would be blunted if the physical barriers and economic costs of transporting goods to and from local markets remain high. The alleviation of these economic constraints, and the equally important easing of the barriers to social intercourse and development would require improvement in access between villages and markets, schools, medical, economic, administrative, and social services, which affect the day-to-day lives of rural people.

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Like most developing areas, sub-Saharan Africa countries face two major rural transport gaps:

1. The rural and feeder roads connecting villages and farming areas to each other and to market centers are usually inadequate, poorly maintained, and costly to use; and
2. Poor and inadequate rural transport services, caused by the lack of intermediate means of transport (IMTs) and appropriate infrastructure for their use, has meant that the carrying of goods between and within villages and between fields, villages, roads, and markets is dependent almost entirely on walking and head- or shoulder-carrying.

By far, the majority of transport and travel activity in sub-Saharan African countries occurs in rural areas. The greater part of transport in rural areas—in terms both of distances and amounts carried—is usually off road. Almost all this transport, in turn, is nonmotorized—in fact, nonwheeled—and dominated by head-carrying by women and children of loads up to 30 kg.

Large efforts at rural transport improvement in SSA have, so far, concentrated on the design and construction of secondary and tertiary roads for motorized transportation. As a result, there are now approximately 700 000 km of recorded rural roads in SSA (Table 1). The condition of rural roads is poor, with about half the network requiring substantial rehabilitation. Even in the countries with the highest concentration of roads, the density of rural roads is low compared with the needs for full economic development. Completed World Bank projects have so far constructed or rehabilitated some 89 000 km and enhanced maintenance over a further 75 000 km of the network (Table 2). The World Bank has estimated that partial rehabilitation of existing rural roads would require an outlay of U.S. $3 billion (*2,3*), which compares with the U.S. $1.7 billion of World Bank–funded rural road rehabilitation over the past 25 years (Table 3). If rehabilitation were to be done over 5 years, the cost (U.S. $600 million per year, with a further U.S. $400 million for routine and emergency maintenance) would amount to about 0.7 percent of the region's GNP of U.S. $149 billion in 1987 (*3,4*).

The rural road networks have a skewed distribution in lengths between countries. For example, Nigeria, Cameroon, and Ivory Coast alone have more than half the rural roads in West Africa, whereas Zaire, Zimbabwe, Madagascar, and Tanzania

TABLE 1 LENGTHS (km) OF ROADS BY TYPE AND REGION (*3*)

Region	All Roads	Rural Roads	Density (m/km2)
Western Africa	430,937	286,425	32
Eastern and Southern Africa	589,943	398,972	36
Total	1,020,880	685,397	34

TABLE 2 RURAL ROAD PROJECT ACHIEVEMENTS IN SUB-SAHARAN AFRICA (*3*)

Period	Roads Constructed or Rehabilitated		Roads Maintained	
	Target Km	Actual %	Target Km	Actual %
Before 1980	49,999	81	28,978	42
After 1980	39,112	72	46,193	49
TOTAL	89,111	79	75,171	43

TABLE 3 RURAL ROAD PROJECT COSTS ($ MILLIONS, IN 1988 PRICES) IN SUB-SAHARAN AFRICA (*3*)

Period	No. of Projects	Loan or Credit Amount	Project Cost	Rural Road Component
Before 1980	79	2,860	6,068	887
After 1980	43	2,408	5,812	863
TOTAL	122	5,268	11,880	1,751

account for more than two-thirds of East African rural roads. Rural road density also varies widely within individual countries. Nigeria also has an average of about 83 m of rural road per square kilometer (individual states' values range from less than 30 to 490 m). Similar ranges were found in Kenya where high rural road densities were associated with provinces with high population and abundant natural resources (*3*).

A recent study (*5*) on food aid and development in the countries of the World Bank's research project "Managing Agricultural Development in Africa (MADIA)" has estimated that the present rural road network of SSA needs to be substantially increased if the full agricultural potential of the region, necessary to support the population, is to be realized. Even if only a part of the extra rural road network required to increase densities were to be built, the annual outlay could double to U.S. $2 billion a year, or about 1.4 percent of SSA gross national product (GNP). These figures are relevant given the 4 percent per annum target for agricultural growth in the region forecast in the LTPS, which translates to about a 1.5 percent increase in GNP. This target for extra agricultural production is unlikely to be achieved if there is no improvement in rural roads and other infrastructure. The needs are great, and although it may be possible to make a viable economic case for an improved and larger rural road network, the concept is not feasible unless specific financial and institutional arrangements for construction and maintenance can be put in place. The question is whether SSA countries can sustain the large network of roads that would result from massive improvements.

The problems caused by the lack of reliable, well-maintained rural road systems are exacerbated by the lack of motor vehicle capacity caused by relatively small vehicle fleets and the poor condition of the vehicles that exist in many countries. The vehicle fleets are small as a result of the generally low importation of vehicles. Data collected from Cameroon, Kenya, Malawi, Nigeria, Senegal, and Tanzania, the study countries of the World Bank's MADIA research project, confirm the low vehicle availability and importation levels. Even Nigeria with the largest share of vehicle imports in the region has shown declining trends. After exceeding U.S. $2.8 billion in 1981, total vehicle import values for Nigeria have decreased 90 percent, accounting for only U.S. $295 million in 1987. The other five countries only showed modest increases up to peak values ranging from a high of less than U.S. $50 million for Malawi and Senegal to U.S. $192,000 for Cameroon in 1987 (*6*).

In addition to these trends in reduced imports, which gradually reduce the size of vehicle fleets, the aging and deteriorating vehicle parks in many countries add to the continuing constraints. In many countries, vehicles have been maintained only by cannibalizing broken-down vehicles for scarce spare parts. Survival and continued use of many vehicles have depended on the ingenuity of good indigenous mechanics in modifying vehicles to the prevailing conditions including the spare parts available. Notwithstanding these remarkable achievements, vehicle capacity and availability remain inadequate. Even where motorable roads exist, transport services are unreliable and infrequent in most rural areas; where available, such services

are for-hire and the majority of rural inhabitants cannot afford them (*7*). This situation clearly highlights the need for alternative approaches to satisfying the rural transport market.

Beenhakker et al. (*8*) pointed out that an improved feeder road may only satisfy 5 percent of local farmers' transport needs, and then only after the surplus has been transported to the road side. This fact suggests that only an extremely small proportion of the total rural transport and personal mobility journeys would benefit from the extension of roads suitable for motor vehicles. Progress in these fields has been slow and expensive, and the transport and travel burden of the poorer rural inhabitants will not be significantly relieved for many decades to come. Without an alternative approach, transport services in rural areas will remain inadequate to sustain acceptable levels of social and economic activity and development.

The transport characteristics have been summarized in three hypotheses that form the basis for discussion of policy issues and options.

Inadequate on-road transport services coupled with the lack of off-road transport, which reduces the timeliness and amount of agricultural goods delivered to on-road transport, seriously constrains the growth of rural economies. Deficient off-road transport also restricts the flow of agricultural and other inputs from the road system to farming villages. Raising the capability of off-road transport could be reflected in much higher returns to rural road investments.

Significant productivity gains can be achieved by low-cost investments in improving off-road transport. These gains reflect both the ability to move more goods more quickly within rural communities and important reductions in the time and drudgery that afflicts transport tasks—particularly those undertaken by women. Reducing transport burdens could, thus, free significant human resources of energy and time for the economic and social development of rural Africa.

The returns on such investments (i.e., for appropriate low-cost infrastructure and IMTs ranging from wheelbarrows to motorcycles) to farm families themselves would be so high that they would be undertaken by private individuals, and the government's role could, therefore, be largely facilitative and promotional rather than requiring direct investment.

RURAL TRAVEL AND TRANSPORT TASK

The rural transport task in sub-Saharan Africa is of far greater complexity than is shown in most traffic surveys. Most surveys are conducted along roadways, and usually record only motorized traffic. They ignore the large numbers of informal means of transport such as pedestrians, bicycles, pack animals, and animal-drawn carts. Motorable roads comprise only a small portion of the rural transport and travel network. There exist vast but unmeasured networks of tracks, paths, and trails linking scattered villages with each other and the fields in which women, children, and men do the bulk of their work.

Predominance of Head-Loading

In many food-growing areas, food has to be carried many kilometers by head porterage either to the village market or roadside points from which vehicles transport it to major consuming centers. For many rural people, a trip on a bus, matatu, or bush taxi still entails a long access walk from the village (*9,10*). Head-loading is frequently performed in conjunction with other farming activities such as weeding and harvesting. Heidemann and Kaira (*11*) report that in Kirinyaga District of Kenya, 70 percent of all journeys made in the rural areas were on foot, with 7 percent by public transport and only 2 percent in private cars. Transport in these circumstances is limited to what can be back- or shoulder-carried or head-loaded to road sides and along rural roads. Riverson and Afele (*12*) also reported that in Ghana, on average, 38 percent of the weight of commodities carried on more than 200 rural roads were by head loads. The proportion carried by head load was up to 90 percent for roads carrying much less than 25 vehicles per day (vpd), but roads carrying 100 vpd and more also registered up to 25 percent of total goods movements by head-loading. The two major reasons given for head-loading along rural roads were (a) lack of alternative means of transport, and (b) lack of funds to pay for the for-hire services when they were available.

Time Used in Rural Transport Activities

Research studies in Ghana and Tanzania (*13,14*) have confirmed that activities involving transport consume inordinate amounts of household time and energy in rural areas. The average total time spent by village households on transport ranged between 1,875 hr per year (for about 80 ton-km in southwest Tanzania, with an average household size of 4.5) and 4,830 hr per year (for about 216 ton-km in study villages in Ashanti, Northern, and Volta regions of Ghana, with an average household size of 11.9).

The time and energy spent on transporting for agricultural and other household activities are major constraints to increased agricultural production or other productive economic ventures. Household transportation, especially for water and firewood, is in fact the major task that competes for household labor time and limits ability to do other things. Internal trips (for water and firewood collection, crop production, and marketing, to local market and grinding mill) represented 73 and 80 percent of total time and ton-kilometers spent on transport, respectively, in the Ghana and Tanzania studies. External trips were those to health facilities, external markets, and outside locations.

Extent and Distance of Head-Loading

In a study (*15,16*) of Ashanti Region, Ghana, it was found that for more than 90 percent of the households surveyed the principal means of carrying goods from the field was by head load. Fifty-six percent of the households used only household labor, whereas 44 percent used hired labor, or a combination of household and hired labor. Motorized transport was used to a significant degree only in one zone where tractors were reported as one of the principal means of transport. The average distance between field and village was 3.9 km, with nearly three-quarters of the distance taken up by footpaths. In only two of the eight zones studied did motorable tracks and feeder roads count for an appreciable component of journeys from

TABLE 4 TIME REQUIRED TO HEAD-LOAD CROPS FROM FARM TO VILLAGE OR ROAD (*9*)

Crop	Kg/ha	Person-days Needed to Headload 1 Acre from Farm to Village	Type of Labor Typically Used for Headloading
Cocoa	900	15	M,F,H,C [1]
Oil Palm	10,000	167	F,H,C
Cassava	10,000	167	F,C
Maize	1,900	32	F,C
Rice	1,500	25	F,C
Plantain	9,000	150	F,C
Cocoyams	7,000	117	F,C
Yams	8,000	133	F,C

1. M = Male, F = Female, H = Hired hand, C = Child

TABLE 5 CHARACTERISTICS OF SELECTED EXPORT AND FOOD CROPS IN GHANA (*9*)

Crop	Marketed Product	Storage Involved	Yield Kg/Ha	Storage Life	Keeping Quality
Cocoa	Dry Beans	Yes	900	6-12 mos.	Good
Oil Palm	Palm Oil	Yes	NA	6-12 mos.	Good
	Palm Kernel	Yes	NA	3- 6 mos.	Fair
	Fresh Fruits	No	10,000	1- 2 wks.	Poor
Maize	Grain Maize	No	1,900	6-12 mos.	Good
Rice	Milled Rice	Yes	1,500	6-12 mos.	Good
Plantain	Fingers	No	9,000	1- 2 wks.	Poor
Yams	Tubers	No	8,000	1- 6 wks.	Fair
Cocoyams	Corms	No	7,000	1- 3 wks.	Poor
Cassava	Roots	No	10,000	1- 2 wks.	Poor

farm to village, and in two other zones, a part of the journey from farm to village was along a main road. Cocoa loads have also been found to be carried an average distance of 5 km from the farm to village or buying centers (*9*).

Based on carrying, each day, two head loads of approximately 30 kg over an average distance of 5 km, it would require 15 person-days to move cocoa (900 kg/ha), and 167 person-days to move cassava and palm fruits (10,000 kg/ha) from a 1-hectare (1-ha) field to a nearby village or the roadside (see Table 4). In general, about 70 percent of agricultural activity involves transport with load consignments of 15 to 150 kg carried a distance of 1 to 13 km for on-farm trips, and load consignments of 15 to 150 kg carried a distance of 1 to 20 km for off-farm trips (*10,13,14*). Head loads of cassava, maize, cocoa beans, plantain, and palm fruits weigh typically 30 kg each (*9*), which is similar to other estimated head load weight range of 25 to 40 kg (*17*). Heidemann and Barth (*18*) present a comprehensive discussion and analysis that confirm these characteristics. If average load sizes and speed of these movements could be increased, rural families would gain great savings in time and energy expenditures for tasks from wood and water gathering to field-to-road transport of produce.

Impact of Transport Constraints on Food Production

Local food supplies from many rural areas do not usually respond quickly to changes in demand (*19*). Although many factors contribute to this effect, some transport-related ones are highlighted. Increases in production of smallholder farmers who dominate African agriculture depend on the assurance of good prices and market opportunities. Market opportunities, in particular, are constrained by poor rural travel and transport services that adversely affect accessibility and personal mobility. Although often unaccounted for, farmers' production activities and costs include costs of trekking between home and field often over long distances, quite apart from the transport requirements for marketing of harvested crops. The proportion of production costs represented by transport and travel costs usually increases rapidly with distance between village and fields. These constraints in off-road transport limit farmers' production to fields closer to the village. Dapaah (*9*) contends that the slow and expensive head-loading, as the only alternative to the limited capacity of unreliable vehicular transportation, as well as the poor keeping qualities of most staple food crops in West Africa (Table 5) are also major factors in the limiting of food production to levels considerably below the potential of rural areas. The extent to which this applies, however, is not quantified and calls for additional study and research on the subject.

INTERMEDIATE MEANS OF TRANSPORT

Indications for Use of IMTs

The time and energy burdens involved in off-road transport represent one of the major constraints to any efforts by smallholders to move from subsistence agriculture to produce more marketable surpluses. As described, agricultural production and consumption within rural areas rarely involve motor vehicles (*10,20*), rely heavily on household labor, and create a need for time- and energy-consuming movements of small loads over relatively short distances. Necessarily small load

sizes, on the other hand, mean that the return to labor—and the surplus it generates—is small, and the marginal cost of increasing production—in terms of time and energy expended—is high.

The trip ranges and load consignments typical of these transport tasks could suitably be undertaken using IMTs. IMTs have higher load capacities than head-loading and the potential for higher travel speeds than walking, and thus significantly reduce the amounts of time and energy spent walking and head-loading. For example, McCall (*17*) estimates that walking about 4 km to a plot reduces time available for fieldwork by about 25 percent, even assuming a generous 8-hr day. As a result, intensive cultivation of small-scale farming crops generally tends to drop off beyond 2 to 4 km (i.e., 1 hour's walk) from home villages. For female farmers, walking to distant farms significantly reduces the time and energy available for domestic household and other economic activities. These constraints to farmer productivity may be reduced greatly by affordable transport options that could facilitate local personal travel and load movements. The experience in other developing regions demonstrates that the introduction of low-cost IMTs, in conjunction with simple, usually community-based, infrastructure improvements, can provide a level of improvement in rural transport that will affect travel for social, educational, health, administrative and recreational, as well as directly economic purposes, that is rarely achieved through road improvements alone.

Potential of IMTs

IMTs are widely used in the developing world, both to improve the efficiency of directly productive tasks, and to serve as a bridge between rural fields and villages and nearby road networks or market towns. The list of IMTs used is impressive and has been widely documented (*7,8,21,22*). Nonmotorized transport ranges from bicycles with or without attachments to animal-drawn carts. The latter often make use of draft animals that are also used to aid cultivation. These human- and animal-powered vehicles provide low-cost and widely affordable means of transport, and their capacity and speed—though limited compared to motorized transport—is far beyond that of head- or back-loading. The design and manufacture of motorized low-cost IMTs have also been documented. However, the geographical coverage of their use is limited (*21*). As a result, some vehicles are used in some countries and not others, or in some regions and not others within the same country.

The design and application of existing IMTs tend to be specific to a country or region of a country. The extensive use of the motorcycle with sidecar, in the Philippines, for example, is one case where local needs appear to be adequately served through a locally developed and accepted solution. Animal-drawn carts have been adopted in some parts of Africa as a result of programs to promote the use of draft animals. Use of bicycles and mopeds is widespread in Burkina Faso, and bicycles are used in Northern Ghana, Uganda, and Rwanda. Kaira (*10*) reports some use of bicycles and animal carts in Kenya; Dixon-Fyle and Relf (*23*) report that in Malawi, the United Nations Food and Agriculture Organization is promoting the use of animal (draft) power for transport, which dates back to the Lilongwe Land Development Project in early 1970s. Reports on India describe extensive use of nonmotorized transport, particularly animal-drawn vehicles using bullocks, buffaloes, camels (in desert areas of Rajasthan),

TABLE 6 PERFORMANCE CHARACTERISTICS AND RELATIVE COST OF SOME IMTs (*27*)

Vehicle	Max. Load (kg)	Max. Speed (kph)	Max. Range (km)	Terrain/Route Requirements	Relative Cost
Wheelbarrow	100	5	10	Flat, narrow path	20
Bicycle	75	20	20	Flat, narrow path	50-90
Bicycle and trailer	200	10-15	15-20	Flat, wide track	90-150
Bicycle and sidecar	150	10-15	15-20	Flat, wide track	90-150
Pack Animal	100-250	5	15-20	Hilly, narrow path	Variable
Animal-drawn sledge (buffalo)	200-400	5	10	Unsuitable for steep terrain	10
Animal-drawn cart (oxen)	500-1500	5	15-20	Flat, wide track	100-180
Motor cycle	100	40-90	100	Motorable path	250-600
Motorcycle and sidecar:	250-500	30-60	60	Unsuitable for steep hills	350-800
Motorcycle and trailer	250	30-60	60	Unsuitable for steep hills	350-800
Single-axle tractor and trailer	1500	15-20	40	Unsuitable for steep hills	1500
Asian Utility Vehicle	1000	60	60	Motorable road or track	3000

Typical values are quoted with variations expected in specific locations. No currency is quoted or intended for the relative costs. The order of cost magnitude is in relation to other values in the Table.

horses, and mules (in urban areas). The type of transport used is closely associated with the nature and type of farming undertaken. Hence, where bullock ploughs are used, bullock carts are predominant, whereas tractors with trailers are associated with tractor-related farming. The performance characteristics of various IMTs are presented in Table 6. A great deal of IMT know-how is available; information needs to be widely disseminated within countries and between countries to ensure more widespread use. However, in each local situation, needs must be clearly identified so that suitable interventions can be made.

WOMEN AND RURAL TRANSPORT

Transport Burden of Women

Households constitute the unit of agricultural production in Africa, and household labor is a most important resource. Any measures to increase the time available to households for productive agriculture or to improve the movement of commodities will contribute to the long-term processes of socioeconomic development (*24*). Hence, reducing household transport time is essential, especially for women, because the most important productive activity in sub-Saharan Africa is small-scale agriculture for which the majority of labor is carried out by women. Head-loading of food crops is typically carried out by women and children. Thus, the transport burden is particularly onerous for women in rural Africa, who play predominant roles both in domestic and economic activities, including the production of food and cash crops (Tables 7 and 8). Women accounted for more than 70 percent of transport time and ton-kilometers carried in Tanzania (*13,14*). The large amount of female and child labor required to head-load food crops from farm to village or roadside have important implications for the attainment of food self-sufficiency in several African countries. McCall (*17*) also reports that women almost exclusively transport water and biomass fuel in addition to the movement of harvested crops from the fields.

Female labor availability in terms of quantity, seasonability, location, labor quality, and incentives, is therefore the key to agricultural improvement (*24*). As stated earlier, much time and energy are spent on transport-related activities associated with rural lifestyles.

Varying Roles of Women

The role played by women in the African transport industry varies from country to country. Hine et al. (*16*) recount that food marketing in southern Ghana is dominated by a large number of independent operators, most of whom are women. They collect produce from rural areas (rural assemblers), arrange for its transport to town (traveling wholesalers), transport the produce (transporters), wholesale the produce in urban areas (nontraveling urban wholesalers), and finally sell it to the urban population (retailers). Throughout West Africa,

TABLE 7 WOMEN'S PARTICIPATION IN LABOR ACTIVITIES: GENERAL AFRICAN SITUATION (*18*)

Production Activities	Percent of Work by Women
Cash Crop Production	30 - 70
Food Production	60 - 90
Food Processing	100
Animal Husbandry	30 - 50
Marketing	50 - 80
Brewing	90
Water Collection	90 - 100
Fuel Collection	80 - 100
Transport of Crops from Field	70 - 90
Household or Community Activities	
Rearing and Care of Children	100
Cooking	100
Cleaning, Washing, etc.	100
House Building and Repair	30 - 60
Communal Farming	50 - 80
Social - Dances, Funerals, Weddings, etc.	50
Litigation Activities	10 - 20
Political Meetings, etc.	10 - 25

TABLE 8 PARTICIPATION IN LABOR ACTIVITIES BY GENDER: A CASE FROM BUKOBA DISTRICT, TANZANIA (*18*)

Average Hours Spent per Adult Household Member in 14-hour Day		
Type of Activity	Male Input	Female Input
Total On-Farm Work	3.10	4.37
Domestic (food preparation, wood, water, child care, etc.)	1.16	3.37
Total Work - including paid employment	6.44	7.90
"Leisure"	4.83	3.41

women are actively engaged in selling food crops and prepared foods and other products in traditional rural and urban markets. Urban market women sometimes prefinance the purchase and transport of food from rural farmers. Within rural areas, the constraints on the rural transport and travel task for women can have two main effects: (a) they have a relatively high workload in other economic sectors (both market and household oriented), which is cut into by transport constraints; and (b) the heavy burden of head loading can cut into agricultural productivity, which also depends on a large input by women.

Potential for Relieving Women's Transport Burden

The extent to which the transport burden on women can be ameliorated will depend on the policies affecting rural development and the role of women in the planning of transport and social services. It will also depend on the willingness of men and women to learn new skills such as bicycle riding for the performance of their duties. In all cases, the transition must be gradual, and will depend on the education, experience, and attitudes of husbands and their female partners. In general, IMTs introduce new roles for husbands that will ultimately reduce the burden on rural women. Experience shows that, usually when improved means of goods transport in the form of pack animals such as donkeys, ox sleds, or tractors have been introduced, men increase their share of transport because of the skills required. However, the issues and the resultant problems must be understood by both genders who must participate equally in the process.

INDICATIONS OF THE NEED FOR CHANGE

IMTs such as bicycles with or without trailers, and animal-drawn carts have proven their worth as labor- and time-saving and productivity-increasing devices in many areas of the developing world. More widespread introduction, in Africa, of proven relatively low-cost IMTs, with appropriate infrastructure aimed at improving access, would provide a valuable complement to road improvements. Improving off-road transport and relieving its onerous burden on rural farm families—especially women—can be expected to increase production and speed transport to rural roads, thus increasing the volume of goods on roads and improving the economic impact of the roads themselves. IMTs, properly developed locally, would complement rather than replace existing motorized transport systems where these exist. They would, therefore, provide an intermediate transitional improvement to satisfy transport and travel needs until improved motor roads and vehicular transport systems become more widespread. The introduction of IMTs is, therefore, thought of as an important first step in upgrading rural transport technology to induce increased economic activity. In effect, the aim is to provide a south-to-south transfer of appropriate technology because these IMTs are in significant use both in Asia and Latin America. In this regard, it is essential to understand the pattern of adoption and development of IMTs and to identify the helping factors and the barriers.

Extensive use of IMTs will also necessitate the construction of suitable paths. Beenhakker et al. (*8*) present specifications for the development of suitable infrastructure for IMTs. These infrastructure design considerations will, however, have to be taken into account in any policies aimed at promoting the use of IMTs. For example, a >2-m-wide unpaved path or trail for pedestrians, wheelbarrows, and bicycles would cost less than 10 percent of the cost of a 6-m-wide all-weather rural road for motorized transport. Such access infrastructure would also be more amenable to locally mobilized labor-based construction and maintenance.

Ongoing Applications in Africa

The potential of IMTs has been shown in some recent donor-assisted investment projects designed to improve personal mobility through the introduction of intermediate transport facilities. These have included bicycles with trailers introduced in Ghana with World Bank assistance, and wooden wheelbarrows and donkey panniers by the International Labor Organization (ILO) in Makete district, Tanzania, with assistance from the Swiss Development Corporation (see Box 1). Reaction of rural populations to these initiatives will do much to determine the interest that rural families show in the potential of IMTs to save time and energy and to raise productivity. It will also highlight their willingness to undertake the investments needed to acquire such devices, and to provide and maintain the usually rudimentary infrastructure needed to operate them efficiently.

Government Roles

These recent projects, also, will help to demonstrate the roles governments could play in promoting IMTs. Actions may range from providing information on such devices to making credit available to purchase them, and could also involve education, technical assistance, or community organization to ensure that appropriate infrastructure is built and maintained. Governments may also have a role on the supply side in promoting the production of IMTs. Evaluation of the extent of IMT use would also form a basis for quantifying the impact of improving rural transport and mobility in general.

Reasons for Slow Development of IMTs in Africa

Africa has lagged behind other developing regions in its adoption of IMTs to increase rural productivity. Kaira (*10*) cites two important reasons for the slow uptake of IMT: attitude and custom (or lifestyle). The attitudinal problems start with a bias against governmental, institutional, and commercial bodies that favor road construction and motor vehicle use as the unique solution to rural transport problems. This bias will only be overcome with a clear understanding of the benefits that can be achieved through relatively low-cost investments in IMTs and rural transport infrastructure. Studies have been proposed as part of the Rural Travel and Transport Project of the World Bank and the Economic Commission of Africa's sub-Saharan Africa Transport Program that will attempt to isolate and quantify these benefits.

Developing IMT in Ghana and Tanzania

In Northern Ghana, field studies have demonstrated that taking into account both speed and payload, a bicycle trailer can increase a person's capacity by at least five times. A bicycle trailer enables the rider to carry up to 440 pounds, and specially designed small handcarts, as much as 330 pounds. These intermediate forms of transport can greatly reduce the amount of time and human energy wasted through back and head loading. Based on these findings the Bank is monitoring performance and productivity.

With International Development Association financing, Ghana's Technology Consultancy Center is promoting the production and use of two forms of intermediate transport—bicycles with trailers and hand-propelled farm vehicles. Vehicle designs have been adapted to local conditions, and demonstration lots are being produced for promotion through local organizations. The third phase of the project establishes credit lines and provides technical assistance to start-up firms to manufacture, assemble and maintain the vehicles. The nurturing of local institutional capacity will provide a sound basis for a longer term strategy to promote intermediate transport in Ghana.

These efforts by the Bank are complementary to similar efforts being undertaken by the ILO in Makete, Tanzania with the support of Swiss Development Corporation. In Makete, local carpenters have been trained to make specially designed wooden wheelbarrows and donkey panniers for use in the hilly areas of Makete district. Demonstration projects have been used to assist marketing, and with the cooperation of the district council, project staff have provided technical advice and supervised local communities in labor-based rehabilitation of earth and gravel roads on a self-help basis. A survey of paths has been undertaken to assess the improvements required for use by wheelbarrows and donkeys.

BOX 1

Technological Degradation or Upgrading

A major obstacle to be overcome is the notion held by many African policy makers that the use of IMTs amounts to technological degradation. However, this perspective can only be justified if IMTs are viewed in the context of motor transport as the only feasible alternative. They cannot be justified when viewed in the context of preponderance of head- and back-loading as exist at present, and with the realization that motor roads and road transport are still decades away from universal availability and use. In addition, the educational curricula for professional engineers and planners have usually been geared to the design and development of high technology. In most countries, not much is done to promote design practices that make the best use of available local resources. It is easily forgotten that transport development in industrialized countries began with draft animals and carts, and other forms of IMTs, and that changes came as economic development increased and incomes rose. On custom and lifestyle, Kaira (*10*) argues that failure of the animal-drawn cart, for example, may be caused by lack of experience in feeding, training, and caring for animals. In some places, climate and existence of pests such as the tse-tse fly may also have contributed to unsuccessful adoption. Local taboos may also have prevented certain segments of the society—such as women—from riding bicycles and tricycles. However, in other parts of the world experience has shown that, with increased levels of education and the appreciation of the potential benefits, this situation may change.

Rural Transport Institutional and Planning Gaps

National transport planning systems in sub-Saharan Africa have often neglected the secondary and tertiary roads that serve rural areas—and largely ignored the off-road transport and travel that make up the bulk of the rural journeys. Possible—in fact, quite likely—synergistic interaction between these two modes of transport has received almost no attention.

A further problem is that the major rural transport and travel tasks, such as the extensive use of nonmotorized transport, including walking with back- or head-loading, is not recognized or taken into account by most planning approaches. The planning approaches focus on roads and their requirements assuming that someone, usually the private sector, will provide the vehicles and services needed for the roads to be used at an economic level. They also assume that rural people can afford the potential motor trips and transport activities predicted. These assumptions are not valid in many rural areas in which the opportunity costs of time and labor for getting to roads are the dominant factors governing, and often limiting, their use.

In addition, the methods and tools applied by many donor and national agencies, sometimes with minor modifications, are dependent on extensive data bases that are not available in most developing countries, and even less so in their rural areas. This paucity of data is a deficiency that continues to create doubts on the applicability of these methodologies for transport investment decisions. Most African planners would rather take their decisions on the basis of local knowledge to satisfy locally defined needs and assume that roads will create the required impact on rural development. The lack of data affects projections into the future for rural roads that generally serve areas with relatively low levels of development.

Invisible and Uncounted Traffic

Rural transport development is based largely on planning and decision-making processes that do not take into account the need for farm families to get more quickly and efficiently between their homes and fields and the rural roads to which they must bring their produce if it is to reach market. Head-loading, though an important cost of production, has usually been excluded from most rural transport evaluations. Part of the gap in data is caused by the fact that even on motorable roads, nonmotorized transport and travel are not recorded or counted. The impact of this would be evident when one examines traffic statistics on rural roads for which all movements

are recorded. For example, recent counts on rural roads in southwest Uganda, where bicycle use is widespread (*25*), indicated that the pedestrian and bicycle counts far outweighed the traffic distribution of motor vehicles on 55 rural road count locations (see Box 2).

Bicycles—The Work Horse of Rural Uganda

The bicycle has become one of the lynch-pins of Uganda's path to rural economic recovery. Although official figures are not available for the total number of bicycles in the country, their existence and intensive use in many rural areas are providing an obvious and visible value to the producers, suppliers and marketers of the country's staple food—matoke (green bananas). They are also used extensively for transport of other commodities and cash crops in most rural areas. The official figures that are available obviously understate the level of bicycle imports. However, the evidence of their widespread and intensive use can be seen from recent traffic counts taken from 55 counting stations on rural roads in southwest Uganda. Total traffic counts at those stations averaged 715 movements per day, with 538 pedestrians (75%), 164 bicycles (22%) and only 15 motor vehicles (2%). The motor vehicle traffic of 15 ADT included 4 trucks, 1 tractor, 2 cars and 8 pick-ups/4-wheel drive vehicles. Minibuses and buses were observed at only 9 of the 55 counting stations.

A separate survey and study of the costs of owning and operating a bicycle used in the transport of matoke revealed that all those surveyed were operating on bad roads and tracks. The bicycles were used for an average of 4.3 days per week during which they made 6.3 trips that averaged 13.2 km, one way. Each bicycle moved a total of 260 ton-km per year at an average revenue income of 1050 Ugandan Shillings (UGSh 375 = US$1.00) per ton-km and an average bicycle operating cost of 336 UGSh per ton-km, leaving the owner with a profit of UGSh 715.0 gross per ton-km. This results in a net return to the operator of UGSh 92.0 per hour for 1,420 hours per year. This excludes the farm-gate value of the matoke which would accrue to the farmer if he were the transporter. The average revenue income includes an unstated amount of profit based on the buying and selling prices of the matoke.

BOX 2

Understanding Rural Transport

A better understanding of rural transport and travel needs and the high returns available from low-cost improvements could lead to the development and definition of policies, planning approaches, and other responses that focus on improving farm-to-field-to-road mobility as a necessary complement to roads investment programs. It could also lead to the adoption of policies that would affect the approaches to infrastructure design, construction, and maintenance and give emphasis to the provision of access, rather than to predefined engineered standards.

This situation calls for a review of the level of understanding of the true nature of rural transport needs, priorities, and planning approaches to adequately consider the true nature of rural transport and travel. Cook (*26*), from a review of research on personal mobility in rural areas, concluded that current rural road evaluation methodologies focus on expected changes in agricultural production, but that the real socio-economic impact of rural road improvements comes about largely through changes in the patterns of personal mobility. Various authors (*8,18,22*) have presented some general guidelines on this subject. There is a great need to synthesize the various experiences and to develop a conceptual approach for planning that incorporates the lessons learned from an improved understanding of the rural transport and travel task and needs that would result from an increased use of IMTs.

NEW APPROACHES TO PLANNING FOR RURAL TRANSPORT

Past planning for rural transport has not given adequate attention to transport services in rural areas. Heidemann and Barth (*18*) point out that there has been too much preoccupation with low-frequency, large-volume, long-distance, high-speed transport operations. The result is a neglect of the real transport problems of the majority of rural population, which is founded in their inability to reduce the everyday burden of moving small volumes of cargo over short distances at low speeds and, preferably, at no extra cash expense. Hine (*20*) also emphasizes that roads cannot be considered in isolation from either the vehicles that are to use them or the factors that determine transport demand. Rather, they must be viewed as a component of a much larger distribution system in which changes in one aspect can cause repercussions on all others. For example, improved storage could lead to a reduced need for all-weather access, while better handling and packaging of easily bruised and damaged crops may reduce the need for a good riding surface.

Two-Part Strategy

The nature of the rural transport problem suggests adoption of a two-part strategy for the provision of transport services in rural areas made up of two complementary systems: (a) conventional motor vehicles along the national road network (essentially all-weather) for moving people, consumer goods and services to an area, and farm products and people out of the area; and (b) traditional and intermediate technology modes on the rural access infrastructure joining villages to distribution, buying and collection centers, depots, and markets. In most cases, however, the national system has been the focus of most attention. Little attention had been paid to the important needs of the upstream second part (*10*).

The planning of rural roads and transport must fit the context of national goals and objectives. Beenhakker et al. (*8*) suggest a comprehensive list of national objectives for rural transport development. These objectives cover social, economic, and political aspects and stress the importance of defining clearly the level of access affordable for rural roads and transport to serve different locations and economic and social func-

tions. He notes that there has been a tendency to overdesign, and that the attractiveness of all-weather roads often leads to their construction in areas in which they are not economically justified and may not even be affordable.

Clear objectives must be followed by formulating strategies for implementation of the planned program. These strategies may cover (a) infrastructure design, construction, and maintenance; (b) resource utilization; (c) technology; (d) institutional arrangements; (e) alternative transport; and (f) potential for an integrated approach. The strategy adopted should suit the financial and technical capability and capacity within the country. Ultimately, the goal is to implement a program that can be sustained within the country. Hence, even with technical and financial assistance from donors, the necessary local inputs—institutions and other resources—must be forthcoming to ensure sustainability in the long term.

Solutions adopted under more realistic approaches would take account of all alternative means of transport (including intermediate means of transport) suitable for the identified rural travel and transport situation peculiar to each country. India has some good lessons to offer in this regard. The needs of villagers and rural farmers and people must be adequately catered for. This could only be achieved by using a bottom-up planning approach that begins with the local transport system and needs as the basis for a continuous planning process. This process must actively involve people at the local level, and the level of sophistication should be increased gradually to suit changing knowledge and understanding of local situations and trends. Technical assistance will often need to be provided to assist at the local level.

Institutional Aspects

Many of the problems and policy issues that have been highlighted in this review can be addressed through a review of government transport policies and planning mechanisms in SSA countries. The pertinent institutional issues relate to integrating transport into rural and agricultural development, enhancing the role of women in development through transport improvements, improving rural transport services, and development and promotion of IMTs and appropriate infrastructure for their use. The broader issues concern the appropriate organizational setup for implementing programs to improve rural accessibility, mobility, and transport, including the involvement of local or village organizations and institutions and nongovernmental organizations (NGOs), and mobilization and use of local resources, including the skills and know-how of local people.

Government commitment and cooperation will be necessary for IMTs to be developed locally and effectively marketed. Suitable standards will need to be set for the required infrastructure, with the necessary legislation, regulations, and institutional arrangements. The improvements in personal mobility and transport will have greater impact on agriculture if they are properly coordinated with the provision of agricultural extension, crops storage and suitable rural markets, and inputs for rural development. In India, this combination of marketing, storage, and transport has been effectively achieved through the operations of a farmers' cooperative union (see Box 3).

India's "Mundi" Grain Regulatory Market

India presents an interesting case where rural market infrastructure is privately provided and coordinated with rural transport services. In Gujarat State, there are regulated rural markets owned by farmer cooperatives that are set up by law and run by a management committee (The Mundi Committee). All traders wishing to operate in the market must register with the committee. No sales are allowed in the immediate vicinity of the Mundi market area. Market facilities provided include sheds, storage, carts for transport, and other basic social infrastructure and services. Farmers without transport can use carts made available at the market to transport their produce. Farmers' produce is brought to the market and sold by auction. The buyer is then required by law to purchase the farmer's produce at the auction price. The farmer has the right to reject the auction price by refusing to sell. The farmer can then store his produce for up to a month in facilities available at the market. With these facilities, the major market and transport constraints to the rural farmer are minimized, and the farmer dictates his own prices.

BOX 3

Implications for Rural Transport Development

Government road and transport agencies are likely to be using almost all available human and financial resources, particularly foreign exchange, in the development, maintenance, and operation of the formal systems of transport—main highways and railways. The evidence is that essential institutional arrangements for rural roads cannot depend on central road organizations because most are underfunded and already overburdened by main road responsibilities, and often far removed from the areas served by rural roads. Accordingly, improving transport and travel systems in rural areas must hinge on distinct institutional and funding arrangements for rural roads, including decentralizing some existing institutions, mobilizing local resources including community and self-help interventions, and using local contractors and technicians. However, guidance and technical assistance from in-country and external sources will still be required to assist and train local people for these tasks. Local communities are presently ill-equipped to maintain roads, and will have to be strengthened to improve their capability for rural roads maintenance. However, until motor roads reach them, maintenance needs would be mainly for infrastructure used for walking and any IMTs introduced and used.

CONCLUSIONS

Evidence of the deficiencies in both the rural road network and the motorized transport services in Africa has been presented. The importance of off-road transport in rural areas and the inadequate understanding of, and lack of attention

to, this essential factor of production and rural day-to-day survival has been confirmed. With the predominance of head-loading and walking as the means of personal and goods movements in rural areas, rural travel and transport, accessibility to and from rural areas, and personal mobility of rural people are gravely restricted.

Although there may be a synergy between rural roads and off-road transport, with each amplifying the economic and social impact of the other, as contended in the first hypothesis, this interaction has not been directly studied or quantified. The relationship between rural motor-road use and off-road feeder networks has important policy implications; if a lack of upstream investment—whether private, public, or combined—is preventing rural roads from reaching their full potential, this should be included in the economic analysis and planning of rural road systems. Governments may also wish to investigate whether relatively minor investments in off-road transport and infrastructure could be reflected in significantly greater use of rural roads.

IMT use would also save large amounts of time and energy and reduce drudgery, particularly for women, as stated in the second hypothesis. A major contribution can be made by planning agencies when they count all IMTs and pedestrians in their traffic planning and monitoring studies. This would fully account for the contribution of IMTs to improving rural accessibility and personal mobility improvement. There has been little study of the production and mobility responses that could be expected with the introduction of affordable IMTs. The basic information gap seems to involve the nature and extent of off-road transportation tasks themselves. Although there is considerable data on amount of labor and time expended on the drudgery of head- and back-loading, there has been little analysis of its effect on production or social pattern in rural Africa.

Another question that has not been adequately analyzed is why IMT usage in rural Africa has lagged so far behind that in other developing regions. Has the major constraint been a generally low level of economic activity, poor communications that have stifled awareness of such devices, market failures, macroeconomic policies that have discouraged such investments, or local and cultural barriers?

A better understanding of the potential economic and social value of IMTs and the low-cost infrastructure needed to make them effective to rural residents—and their stimulative effect on production and road transport—is clearly needed to test the assumptions made in the first two hypotheses. Thereafter, transport planners will address the question of how best to stimulate investment in productive intermediate transport devices at minimum cost—in both financial and manpower terms—to national economies. Some promotional and infrastructural work will obviously be needed, but governments will find a path that is stimulative and facilitative without requiring significant allocations of scarce resources. When IMTs have been introduced in the past and used in the transport system, private individuals have usually taken up the initiative to develop them further, and to reap the benefits therefrom as suggested by the third hypothesis.

The development of institutions for the improvement of rural accessibility and personal mobility including the introduction of IMTs would require different approaches to suit particular country situations. But, for all countries, the primary requirement for the success of new initiatives is a clear understanding, by government and local officials, of the policy issues involved and a commitment to pursue solutions applicable to local situations. Once the relevant issues are accepted and the required commitment is attained, it will be necessary to identify suitable existing institutions, parastatals, nongovernmental organizations (NGOs), or private entrepreneurs for the development and promotion of new IMTs. Successful promotion will also depend on the government's creating the enabling environment for the participation of the private sector. In most countries, this process will require understandings from, and policy review and initiatives of, ministries or agencies responsible for agriculture, rural development, planning, roads or public works, local government, transport, and industries.

These concerns are currently being addressed in the Rural Travel and Transport Project (RTTP) of the World Bank and ECA's sub-Saharan Africa Transport Program. The objectives of the RTTP are (a) to develop and disseminate improved policies to plan, finance, build, and maintain rural roads; (b) to recommend approaches to the improvement of rural transport services, and to the adoption of intermediate technologies to increase personal mobility and agricultural production. The specific recommendations are being developed through village level travel and transport surveys and country case studies that focus on (a) analysis of rural transport and travel patterns by gender in a variety of agricultural, social, economic, or transport environments over a given time frame; (b) assessment of past experiences in, and prospects and means of, introducing and developing intermediate (low-cost) means of transport (IMTs), including a determination of the institutional requirements and functions and the external constraints to IMT use; and (c) assessment of the impact on the role of women in improving accessibility by introducing IMTs.

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Policy Foundation for Good Roads in Sub-Saharan Africa

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A well-maintained road network is of paramount importance to the development of sub-Saharan Africa. Practically 90 percent of land commerce is dependent on roads. Significant achievements have been made in expanding the road networks across the continent. But, building the infrastructure of institutions and trained manpower to maintain them has proven difficult. Ineffective maintenance has led to widespread and accelerating road deterioration—amplified in recent years because the large number of roads built in the 1960s and 1970s have reached the end of their useful lives and need rehabilitation or reconstruction. The road maintenance and rehabilitation problem has been exacerbated—as are almost all the region's problems—by the current economic difficulties in sub-Saharan Africa. Roads, however, are so much an engine of growth that making economically justified expenditures to ensure their continued serviceability deserves priority attention in government spending. There has been a growing realization that road building and road maintenance are not in balance. Policies that favored construction over maintenance clearly have to be changed in favor of approaches that look at road expenditures as a whole within a policy framework that enhances transport availability at minimum overall social cost—to government road agencies, and, more important, to road users. The state of road networks in sub-Saharan Africa is reviewed and a classification of countries relative to future road development policies is proposed on the basis of road conditions and past experience with road maintenance. Three critical policy issues are identified in terms of achieving well-maintained and serviceable road networks in sub-Saharan Africa: (a) a national commitment to adequate and reliable financing and budgeting for roads, (b) accountability in the use of public funds for roads in terms of tangible physical performance, and (c) institutional reform and provision of incentive mechanisms to improve efficiency. These policy issues are amplified and recommendations on appropriate funding, planning and programming mechanisms, improved management of road maintenance operations, and institutional reform and development of human resources are provided.

The sub-Saharan Africa road networks (excluding roads in the Republic of South Africa) have an aggregate length of over 1 million km, with about 110,000 km of paved roads. These networks have a low spatial density (about 5 km per 100 km^2 of land area compared with 10 to 30 km per 100 km^2 in Latin America and Asia) and over 95 percent of the road system consists of low-volume roads, i.e., with an average daily traffic (ADT) of less than 400. Between 1971 and 1987, the total length of the road networks in sub-Saharan Africa increased by over 200 000 km, with the paved roads increasing by over 55 000 km (Table 1). But, creating the necessary infrastructure of institutions and trained manpower to maintain these networks has proven difficult. Ineffective maintenance consequently has led to widespread and accelerating road deterioration. Two main factors have amplified this problem. The pace of road building peaked in the early 1960s and 1970s, creating thousands of kilometers of highways that are now reaching the end of their design life at the same time. The turndown in regional economic growth has meant that this bulge in the need for road rehabilitation and replacement has coincided with a severe cutback in the financial resources needed for the task (*1*).

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THE ECONOMIC BACKGROUND

The plight of roads in sub-Saharan Africa, however, is not independent of the general economic situation in the region. The sub-Saharan countries are confronted with negative economic growth, declining agricultural production, and a heavy burden of external debt. In most countries, gross domestic product (GDP) and income per capita have been stagnating or declining for more than a decade. In 1987 alone, real GDP per capita declined 4.7 percent, whereas population growth continued at a rate that adds 20 million people each year. The net flow of capital to the region declined from an average of U.S. $14.0 billion a year during the period 1980 to 1982 to U.S. $8.5 billion a year during the period 1984 to 1986. Total external indebtedness now exceeds U.S. $100 billion and debt service, even after rescheduling, consumes about 30 percent of export earnings. Per capita investment in real terms declined by half during 1980 to 1987, while imports, in constant dollars, fell by 35 percent. As a result of all these factors, the average African is poorer today than in 1970. This situation is clearly unacceptable and hard policy choices must be made by African governments to reverse the trend.

The transport sector—particularly roads—is a critical element in the strategy to increase agricultural production and incomes in sub-Saharan Africa. There are telling examples where transport bottlenecks have put the brakes on growth, especially in agricultural production. The overarching objectives for transport policy can be captured in two words: efficiency (and thus lower costs) and sustainability. Nowhere are policy reforms more needed than in the roads sector. If farmers and manufacturers are to take advantage of overall reform initiatives they must have a dependable road system. Without efficient transport—and in sub-Saharan Africa transport means, more than anything else, roads—there can be no supply response to support renewed economic growth and sustainable development.

TABLE 1 EVOLUTION OF SUB-SAHARAN ROAD NETWORKS[a]

	1971 (km)		1987 (km)	
	Total Road Network	Paved Network	Total Road Network	Paved Network
Eastern and Southern Africa	367,800	22,400	503,000	42,400
Western Africa	341,300	30,700	430,900	67,000
Sub-Saharan Africa	719,100	53,100	933,900	109,400

a/ Excluding Angola, Comoros, Djibouti, Mozambique, Namibia, and Zimbabwe for which 1971 data were not available. Also excluded is the road system of the Republic of South Africa.

TABLE 2 ROAD CONDITIONS IN SUB-SAHARAN AFRICA[a]

	Percent Good		Percent Fair		Percent Poor	
	1984	1988	1984	1988	1984	1988
Eastern and Southern Africa						
Paved Roads	42	50	32	33	26	17
Unpaved Roads	42	28	30	30	28	42
Western Africa						
Paved Roads	52	54	23	17	25	29
Unpaved Roads	20	19	36	34	44	47
Sub-Saharan Africa						
Paved Roads	47	52	27	25	26	23
Unpaved Roads	33	29	32	32	35	39

a/ Excluding the road networks of Angola, Cape Verde, and the Republic of South Africa.

THE STATE OF ROADS IN SUB-SAHARAN AFRICA

In the immediate post-colonial era, many sub-Saharan countries gave priority to road construction in their development programs; maintenance and creation of effective road administrations, however, received little attention. Foreign donors recognized the priority of road transport in the new nations and gave large amounts for road development. But they offered much less to create the institutional and financial resources needed to protect these investments. The result was a burden of road maintenance that increased much more quickly than the ability to carry it. Other factors have accentuated the problems. The rapid buildup of road networks has resulted in networks with a skewed age distribution, which concentrates major maintenance and strengthening requirements in cyclical periods, periodically requiring a several-fold increase in maintenance outlays, which are often beyond the financial capacity of most sub-Saharan countries.

An assessment of main road conditions on the basis of limited data available for 41 countries shows that in 1988 nearly one-quarter of paved roads were in poor condition and a further quarter in fair condition. The situation is even more serious for unpaved main roads, of which nearly 40 percent were in poor condition and an additional 33 percent were in fair condition. Roads in fair condition are assumed to require rehabilitation involving surface treatment or overlay (paved roads) and regraveling (unpaved roads), with selective repair. Roads in poor condition are assumed to require reconstruction. A subregional breakdown is presented in Table 2, based on country-specific network information assembled and reported by Mason and Thriscutt (*2*). The indications are that in spite of great efforts made by governments and aid donors, it has only been possible to barely stabilize the situation. Although there has been improvement in the condition of paved roads, particularly in Eastern and Southern Africa, the condition of unpaved networks worsened over the period 1984 to 1988.

Routine and preventive maintenance based on careful monitoring of traffic loads and pavement conditions can protect and prolong the life of important national investments in roads. A comparison of costs for preventive maintenance and reha-

TABLE 3 DISTRIBUTION OF ROAD EXPENDITURES: CONSTRUCTION VERSUS MAINTENANCE FOR 19 SUB-SAHARAN COUNTRIES

	Percent Distribution of Road Expenditures	
	1981-82	1986-88
New Construction/Improvements	66%	58%
Reconstruction and Rehabilitation	13%	17%
Resurfacing and Routine Maintenance	20%	25%

bilitation illustrates the point. Routine maintenance can be done for a cost of U.S. $300 to $1,500 per kilometer per year and periodic maintenance every 3 to 12 years for about U.S. $25,000. When maintenance is neglected, the eventual cost of rehabilitation or restoration is about U.S. $30,000 per km for unpaved roads and U.S. $200,000 per km for paved roads. These amounts represent a 2.5- to 8-fold increase in expenditures. Also, failing to redirect spending to maintenance, 3 to 4 km of potentially good roads are lost for every kilometer of new construction.

The economic consequences of neglecting maintenance are primarily borne by road users in the form of increased vehicle operating costs (VOCs). For example, the difference between a good road and a poor one may lead to increases in VOCs of 20 to 40 percent—an amount that is from 3 to 10 times the cost of the road itself. Thus, a dollar reduction in road maintenance expenditures can result in an increase of U.S. $2 to $3 in VOCs with a high proportion of these cost requiring foreign exchange for spare parts, tires, and replacement vehicles. In reality then, reducing expenditures on road maintenance will not serve the intended purpose of releasing resources for higher priority sectors. Spending a dollar less for road maintenance will perhaps release a dollar for higher priority expenditures such as on education or health in the public sector, but road users, mostly in the private sector, will have two or three dollars less to spend on essential private consumption. Thus insufficient spending on roads and their maintenance exacts hidden costs several times the expenditures on roads.

The governments of the region could save some U.S. $1.2 billion (of which perhaps U.S. $900 million is in foreign exchange) in road expenditures a year, nearly 0.85 percent of regional GNP, through well-targeted preventive maintenance programs. Another U.S. $1.2 billion would accrue as benefits to road users through reduced wear and tear on vehicles as a result of improved road conditions. These figures do not account for the costs imposed on agriculture and industry by increased transport costs, lengthened travel times, or the delayed pace of development. About U.S. $1.2 billion is needed annually over the next decade to rectify the past neglect of road maintenance and restore all priority roads to good condition. These financial requirements are based on detailed estimates prepared by Mason and Thriscutt (*2*), and account for restoring and maintaining only the high-priority road network in sub-Saharan countries.

The bias towards new investments is still evident in the road expenditures of sub-Saharan countries, although there has been a discernable shift from new construction to rehabilitation and maintenance during the 1980s, as presented in Table 3 (*3*). A number of countries, mainly in Western Africa (Benin, Ghana, Nigeria, and Togo), have belatedly recognized the need to increase allocations for maintenance and rehabilitation.

As shown in Figure 1, countries in sub-Saharan Africa can be placed in four groups, based on their records of road condition and on past experience of their maintenance performance (*1*).

- Group 1: Botswana, Cote d'Ivoire, Djibouti, Malawi, Niger, and Zimbabwe. These countries have demonstrated a substantial commitment to maintenance, and have the ability to implement their road programs, and raise or attract adequate resources for the highway sector. These countries should have little difficulty in restoring—or preserving their roads in good condition, in some cases, as soon as 1995. They have already shown their ability and willingness to put sufficient resources into road maintenance; continuation of the same policies, sometimes with additional external funding, will be needed.
- Group 2: Benin, Burundi, Central African Republic, Ethiopia, Lesotho, Mali, Madagascar, Mauritius, Rwanda, Swaziland, and Togo. These countries have shown a growing commitment to road maintenance but have yet to build up adequate capacity for implementation. Efforts to improve maintenance look promising, and sufficient resources to undertake the needed maintenance could probably be obtained if spending priorities were adjusted accordingly. These countries should be able to restore their roads to good condition by the end of the century if they can strengthen their commitment to improved maintenance even though considerable technical and financial assistance will be needed in some cases.
- Group 3: Cameroon, Congo, Gabon, Kenya, Nigeria, and Senegal. These countries paid insufficient attention to road maintenance in the past, despite having the capacity and financial resources to contain deterioration of their roads. These countries have the capacity to restore their roads by the year 2000, and could raise the finance required, but they will not achieve that target unless there is a significant change in the priority accorded to road maintenance.
- Group 4: Angola, Burkina Faso, Chad, Equatorial Guinea, Gambia, Ghana, Guinea, Guinea-Bissau, Liberia, Mauritania, Mozambique, Sierra Leone, Somalia, Sudan, Tanzania, Uganda, Zaire, and Zambia. These countries have experienced severe problems in building up institutional capacity and attracting funds for road maintenance. Some of their networks are severely deteriorated, and substantial amount of rehabilitation will be required. These countries will probably not be able to restore their networks to good condition

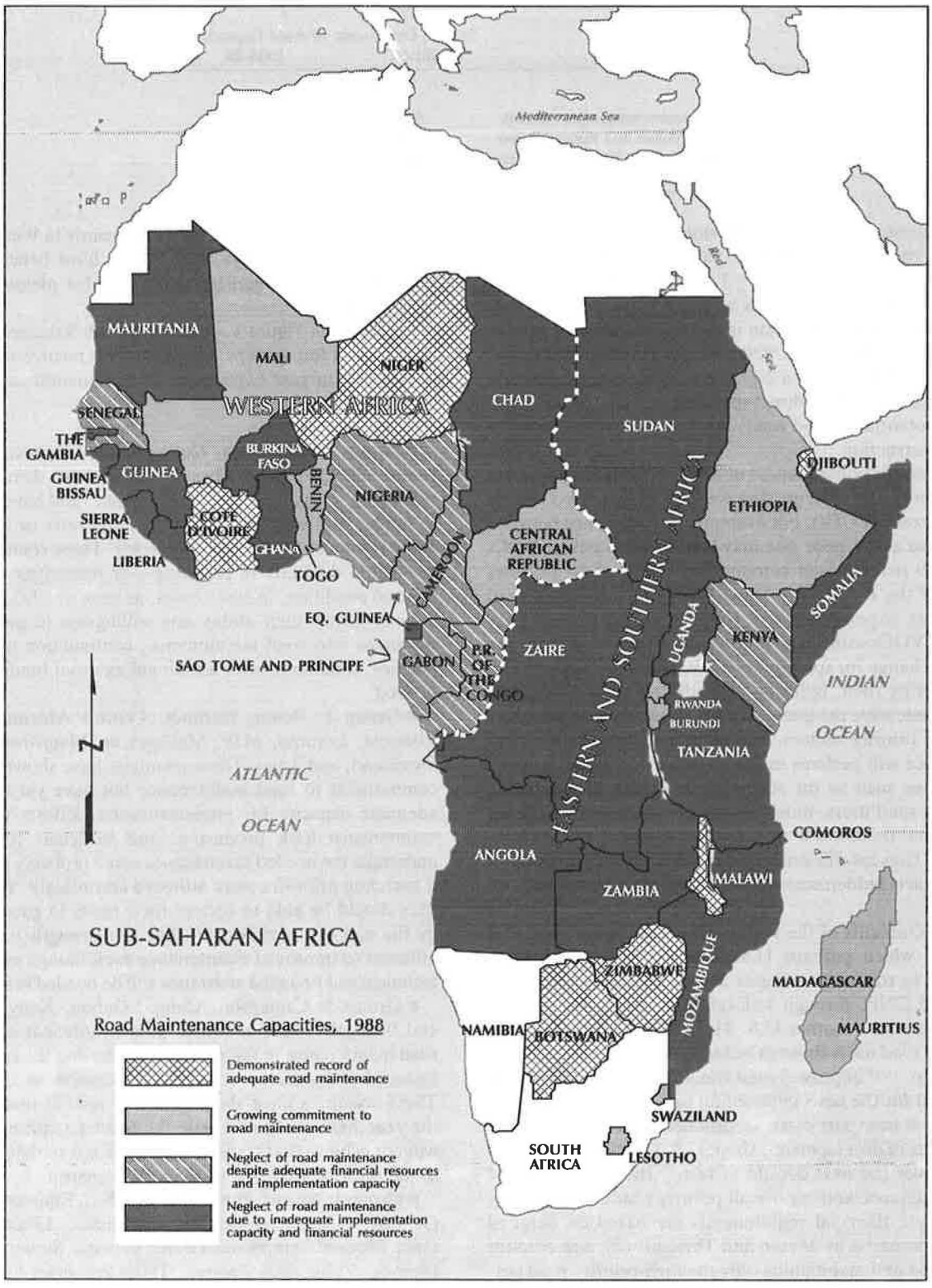

FIGURE 1 Classification of sub-Saharan African countries according to road maintenance capacity.

by the year 2000 because of their economic condition and the size of the backlog of maintenance and rehabilitation. They will have to take difficult decisions concerning priorities of rehabilitation and maintenance that will best serve their short- and medium-term needs, including concentration of maintenance effort on a core road network.

A review of road expenditures in sub-Saharan Africa countries show that some 60 percent (Groups 1, 2, and 3) have been spending (at least over short periods) about 1 percent of their gross national product (GNP), which is enough to restore and fully maintain all their essential roads. In these countries, a redirection of effort away from new works to better maintenance will be essential to avoid road failures on a large scale.

For the remaining 40 percent of countries (Group 4), even stopping all new works until the year 2000 would still require higher levels of road financing than they have previously had. Whether these countries can plan for and manage larger amounts—and whether the donors are prepared to provide extra aid—will depend on the organization and efficiency of their roads administrations.

To summarize, the maintenance and restoration of sub-Saharan Africa's essential road networks would cost about U.S. $1.2 billion a year over the next decade. This is a large amount of money, but it offers potential national annual savings of twice that magnitude; thus, for every dollar spent on maintenance, governments would save one dollar by avoiding reconstruction costs, and road users would save another dollar in reduced wear and tear on their vehicles. In addition, by carrying out this program of restoration, by the year 2000, most of the essential network would be in serviceable condition, whereas if current trends continue, most of it will have to be rebuilt at high cost.

KEY POLICY ISSUES IN ROAD MAINTENANCE

Past policies that have favored construction over maintenance must be changed in favor of approaches that look at road expenditures as a whole within a policy framework that enhances transport availability at minimum overall social cost—to government road agencies, and, more important, to the road users. The requirements for improved maintenance in sub-Saharan Africa can be disaggregated into three critical issues.

The first issue is a national commitment to maintain a serviceable roads system. A critical requirement is a broader financing and budgeting perspective that sees road building, maintenance, and rehabilitation as a unified process within a multiyear perspective. This perspective recognizes it is not enough to allocate funds for roads—that a real commitment includes dependable flow of funds to allow roads departments to effectively plan and implement their programs.

A second issue is accountability. In the absence of public pressure and lacking a clear understanding of the seriousness of the problem, few governments have given road maintenance a high priority in their budgets. The urgency of the situation has not always been fully appreciated by all donors and lending agencies, some having been readier to provide funds for new construction than for maintenance and restoration. New construction has sharp political visibility, maintenance little glamour. Inadequate maintenance in developing countries has various causes, but institutional failure is the only explanation for its wide extent. At the heart of this failure is the absence of public accountability (*4*).

Accountability should also be turned outward towards the road users who are most immediately and directly affected by road conditions. It is the users, rather than maintenance organizations themselves who bear the immediate costs of poor roads. There is a vast public constituency in Africa for good roads; it should be encouraged to organize as a monitor of national road systems and an advocate for their improvement.

The third major issue is the need to adapt, improve, and reform institutions so they have both the mechanisms and the incentives to increase their efficiency. The staff and managers of those institutions will need new skills to carry out these tasks; development of human resources must go hand-in-hand with the development of sector institutions.

AN AGENDA FOR POLICY REFORM

The main policy objective is to identify possible actions to improve maintenance in the face of funding and organizational constraints. A repetition of past trends will result in poorly serviceable road networks with all the attendant costs and constraints on economic development. To restore the entire network would be beyond the ability of most countries, and may not always be the best use of resources, given the needs in other sectors. Governments will need to show a strong commitment to better road maintenance by putting in place the policy and institutional changes necessary for improved maintenance—in particular the revision of road budget allocations in favor of maintenance and rehabilitation; and restoration of the essential road network to good condition by the year 2000. Policy reforms aimed at improved road maintenance fall into three broad categories: (a) funding, planning, and programming; (b) operation and management; and (c) institutional reform and human resource development.

Funding, Planning, and Programming

The reason most often given for low standards of road maintenance is that there is not enough money, but this is only true for about half the sub-Saharan African countries. In the others, past expenditures on road maintenance and construction could have financed maintenance and road rehabilitation adequately. In some countries, although budgeted funds are adequate, they are not released promptly. As a result, work has to stop, sometimes for several months. This is one reason why budget allocations are underspent although the need and means for incurring expenditure may both exist.

The three areas basic to improving funding, planning and programming for more effective road maintenance are (*3,5*) as follows:

1. Developing a unified framework for planning and programming public road expenditures over the entire network, whether recurrent, capital, or aid-financed, in particular by programming maintenance on the basis of monitored road conditions and measurable physical outputs;

2. Funding and budgeting procedures that ensure the funds are available as and when required, and that they respond to changing circumstances; and
3. Financial and technical audits to foster accountability.

Unified Planning and Programming

The goals of roads departments must be set in terms of well-articulated outputs. Planning, programming, and budgeting should focus on improving roads in terms of precisely defined and rigorously monitored physical conditions. This process can only be achieved if there is in place an information system that tells planners both the state of the roads system and the results of previous maintenance interventions and expenditures. Successful experience with road management systems in Ethiopia, Ivory Coast, Malawi, Niger, and Zimbabwe suggests that most sub-Saharan countries can muster the resources and skills to install basic information systems to meet their road management requirements.

The first step toward producing a credible roads program, that can win the commitment of governments, transport customers, and foreign donors, is an assessment of all road expenditures—whether capital, current, locally funded or aid-financed—under a unified programming and budgeting framework. In almost all countries, such exercises will demonstrate that road maintenance and reconstruction have economic returns several times that of almost all new construction. In addition, output-based programming provides a basis for monitoring and auditing performance besides allowing the establishment of objectives to guide road agencies in their operations.

As indicated earlier, road expenditure options cannot be competently assessed unless a management information system is regularly providing the roads authority information on road conditions, traffic, and costs for rational decision making. One of the most useful features of such systems is that if funds are insufficient to finance an adequate level of maintenance, a revised program can be drawn up that will limit road deterioration and user cost increases to a minimum within the funds available. At the same time, the total system costs in terms of future road rehabilitation and increased user costs can be estimated, so that the government is made aware of the consequences of underfunding maintenance.

Funding

Maintaining a road network requires that resource flows be regular and dependable as well as adequate in amount. In the past, the expansion of sub-Saharan road networks, often supported by international aid, has not taken sufficient account of this. Improvements in collecting, allocating, and using funds are obvious measures, but even then regular funds for road maintenance may not be assured because of programming and bureaucratic inefficiencies.

Earmarking road user taxes, such as fuel taxes and tolls, with payments made directly to the account of the roads organization rather than through the treasury or finance ministry, is one way of increasing dependability. Alternatively, a special roads fund may be created into which some or all road user charges and tolls are placed. Either of these mechanisms gives the roads organization some long-term assurance that funds will be reliably available. Thus, a multiyear road rehabilitation and maintenance program can be planned, if necessary with donor cooperation, with the knowledge that funding is secure (*5*). A number of sub-Saharan countries such as Ghana and the Central African Republic have successfully implemented special road funds targetted at improving road maintenance.

If sufficient resources to maintain the national road network cannot be mobilized, a country may have only two options—each of them only a poor second best: continued deterioration of the entire road system, or a reduction of its network to a size that is affordable. To avoid deterioration of the whole network, when funding is uncertain in spite of all efforts, the best remaining choice may be to plan a core maintenance program that preserves essential roads in the best way possible under the given economic conditions.

Performance Audits

Independent financial audits, operating free of ministerial control, and responsible only to the legislature or the chief executive, are accepted parts of most governments. Financial audits, however, normally check only that funds have been spent for the purposes intended as specified in the budget. There is no check that value has been obtained for the funds expended. The concept of productivity and quality of output is not part of the financial audit's brief, unless the faults are glaring. A technical audit, on the other hand, is concerned that the design of works is correct, that the materials used were appropriate, and that work was done according to accepted standards. Obviously, it is convenient and desirable that financial and technical audits are integrated in a unified performance audit. A technical audit should also be free of administrative control. This means that it should have its own facilities, or at least its own independent staff who may use the road authority's laboratories and equipment. Although the road organization may take the lead in setting up a technical audit, the agreement of other ministries and legislature may be needed (*3*). Although few developing countries have implemented performance audits to monitor road development and maintenance works, such controls are common in education and railway organizations.

For a technical audit to be effective, technical specifications are necessary, just as financial audit requires government financial regulations. It is desirable for many reasons, apart from audit, to work towards common specifications for all government work. As the roads organization is usually the most active construction ministry, it is appropriate for it to take the lead in suggesting uniform contracts and specifications.

Operations and Management

Effective and efficient management of operations is crucial to addressing sub-Saharan Africa's road deterioration problem. More effective management that produces demonstrable improvements in both the condition of roads and the efficient use of resources is also a key to creating the political com-

mitment necessary to increase attention to—and funding for—maintenance. It can also be a powerful tool for saving money, particularly foreign exchange.

Most road maintenance operations in sub-Saharan Africa are run by large establishments operating by force account with geographically widespread operations that rely heavily on equipment. Maintenance operations planning and execution often overlap, and information systems are not adequate for management to track the quantity and quality of performance or assess and prioritize needs. Maintenance managers have also been overburdened by the need to keep up huge—and often incompatible—equipment fleets and supervise large and far-flung work forces. Improving information flows and streamlining management tasks are perhaps the two key factors in improving management efficiency.

The key policy recommendations covering operations and management are the following:

1. Increasing accountability by separating planning and monitoring functions from works execution,
2. Adapting the size of the mechanical equipment fleet to reflect maintenance needs and increasing the efficiency of equipment use by relying on private vendors for equipment maintenance and servicing,
3. Spinning off management burdens through increased use of contractors, and
4. Adopting appropriate technology and using local resources for self-reliance and sustainability.

Accountability

As discussed earlier, several steps can be taken to improve accountability in road maintenance operations. One is structural—separating monitoring and planning from execution can strengthen both functions. The rationale for such a separation is similar to that commonly found on the financial side of almost all government operations—those who carry out tasks seldom have the time, the training, or the objectivity to judge either financial or operational performance (*4*). Although separation of operations from planning and monitoring can take many forms, a common approach is to recast the organizational structures to concentrate on supervision and monitoring while spinning operational functions off to contractors.

Efficiency of Equipment Use

Most countries will benefit from reducing and consolidating their equipment fleets to a size and composition they can manage efficiently within the capabilities of available staff, funding, and support services. A smaller fleet with higher availability and utilization rates will save money while increasing productivity and reducing unit costs and management overhead. Equipment maintenance itself can often be consolidated by creating equipment pools as separate single-purpose and self-accounting management units from which the roads authority can hire plant (*6*).

Outside firms can also be used to reduce a roads authority's equipment maintenance burden. Plant and equipment can be hired from private sources, or private firms can be contracted to maintain equipment. In countries in which the private sector is not yet capable of entirely taking over such tasks, this process can be combined with pooling some plant maintenance within the roads authority. This combination can have an added advantage of creating a business environment in which the private and public sector compete to foster efficiency.

Private firms may require strengthening to play an expanded role in contracting and equipment maintenance, and governments may well find it economical to provide training in the requisite skills to build up their capability. Standardization of equipment should also be sought for, at least, the road authority's fleet. Many countries need to institute firm policies and the discipline to maintain fleets of standardized equipment. Aid donors should assist in this effort by coordinating their support to individual countries to aid standardization and by providing foreign exchange for spare parts. Streamlining national procurement and supply regulations can also enhance equipment management efficiency.

Use of Contractors

If local contractors can be mobilized to undertake road works, their use can increase efficiency and reduce costs. Market and competitive forces tend to act more strongly on private firms—especially small ones—than on public sector agencies. Their objectives are usually simple survival and profit and are not clouded by political considerations, and their relatively small size increases their flexibility. These factors motivate them strongly to use staff efficiently and maximize the use of labor rather than capital equipment to conserve resources.

Stable markets and supportive policies are among the most important factors in encouraging development of local contracting industries. If contracting firms know they can expect work to be regularly available, they can invest in equipment and manpower that will strengthen their capabilities. Governments should consider offering training programs to private contractors to increase the skills pool.

Reviewing and, in many cases, simplifying contract management procedures such as contract documents and specifications can encourage firms to enter the market, as can improvements in the timeliness and reliability of payment procedures. Increasing use of private firms will increase the contracting authority's supervision and quality control responsibilities (*7*).

Appropriate Technology and Local Resource Use

Although road maintenance techniques and organizations in most regional countries have been geared to equipment-intensive techniques derived from construction and maintenance practices in developed countries, constraints on resources and the availability of under-employed or unemployed labor will often make the adoption of labor-intensive maintenance techniques economic.

Where labor is available and can be monitored at daily wage rates of up to U.S. $4.00, contractor or community- or village-based maintenance crews can provide effective low-cost road

repair and maintenance. Considerable political commitment to using local resources such as labor exists in most countries, but this needs to be translated into practical administrative and technical programs. In most cases, it is necessary to phase such programs in through pilot projects of 2 years or more. Decentralizing the supervisory, administrative, contracting, and payment authority required for increased use of labor-based techniques can often increase the efficiency of such operations. This improvement should be accompanied by staff training, and the setting of standards for labor-intensive road works (tools, equipment, and road materials). The availability of local materials—such as timber for bridge-building—can enable significant foreign exchange savings. There is also much scope for adapting design standards to maintenance operations to take advantage of local capabilities and local materials, thus saving foreign resource inputs.

Institutional Reform and Human Resources Development

Institutional and human resources development has been one of the most difficult areas in which to make permanent improvements. Establishing self-sustaining institutions that can manage road maintenance efficiently or use external resources effectively has been difficult. Institutional problems with road maintenance arise because of the inherent managerial complexity, and the geographical dispersion of maintenance operations that require a territorial organization. Institutional progress has been slow despite training and institutional development programs of long standing.

Road maintenance has been only one of many claimants on the region's overstressed education and training facilities. Specialized personnel remain in woefully short supply. It is generally agreed that a minimum of about 30 graduate level engineers and scientists per 10,000 people are needed to service a modern economy. In most sub-Saharan African countries, the actual proportion is less than one-third of this number.

There are many issues, but they may be summed up under two main heading:

1. Institutional reform to increase the efficiency of road maintenance and rehabilitation, and
2. Better management of human resources through motivation, adequate incentives, and efforts to develop a stronger and more competent workforce.

The build-up of a competent core of managerial and technical manpower that can perform effectively in support of institutional objectives is essential. There are two main approaches:

- Create a personnel unit within the road organization to strengthen its capacity for manpower planning, recruitment, training needs analysis, organization of training, management of promotion and career development schemes, and administration of compensation and benefits; and
- Strengthen the ability of line managers to utilize, supervise, motivate, and develop their staff effectively.

To administer these functions effectively, line management in particular will need to acquire management skills such as how to set objectives, allocate work, delegate and follow up, set standards, establish accountability, and provide subordinates needed on-the-job training, recognition, and feedback. The two sets of responsibilities (personnel and line management) should be integrated into a single long-term manpower development and retention program. To effectively implement such a program, roads agencies should introduce changes in hiring, promotion, and termination practices to strengthen motivation, retention of talent, accountability, and performance.

When free to operate semiautonomously (but not only in such cases), the road agency can hire only those people that it actually requires to do its job. It will not become financially overburdened with staff on the payroll that it does not really need. Cuts of unnecessary staff can free money for operations, supplies, and equipment and can improve effectiveness and efficiency in the use of remaining staff. This process enhances the motivation and retention of competent people. Staff can be better managed and utilized because funds become available to secure the equipment and supplies necessary for them to do their jobs. Because staff obtain the means to carry out their responsibilities, they also tend to become more motivated. Agency semiautonomy from the civil service can also allow management to promote staff on the basis of merit and accomplishments rather than just seniority. Staff that do not perform can also be more easily removed from the payroll. This kind of management of human resources in terms of hiring and firing and career advancement is an important tool for increasing the internal accountability and motivation of an organization.

Another approach to improve staff performance is through an organization's compensation and benefits system. A semiautonomous agency has more flexibility to pay people at the level required to retain the kind of talent it needs, in competition with the private sector. Even without autonomy, a road agency can institute innovative compensation schemes such as pay directly linked to quantities and quality of outputs and results produced (*8*).

Effective manpower utilization also requires a good system of position descriptions, management by objectives, appropriate workload allocations, and effective staff supervision. These are basic personnel and work management tools. Good utilization of manpower requires appropriate workload and responsibility allocations within units and within the organization. Within a unit, this is the unit line manager's responsibility. Within the organization, any need for overall organizational restructuring should be identified and addressed by general management.

It is also important for an agency to identify training needs, and develop mechanisms to deliver the required training in essential and specific job-related skills. For training and other developmental actions to be effective, they need to take place in a context of improved manpower management and utilization. It is important to remember that if manpower management and utilization are not operative, staff that receive training and are not effectively utilized might opt to leave the organization. If they stay, they may tend to stagnate and not develop in ways that are practical and useful for the organization.

When strong manpower management and utilization are in place, in-service training schemes make an indispensable contribution to staff development. In addition to the educational qualifications and experience that staff bring to the job, they also need to develop additional job-related specific skills and specializations. Careful training needs analyses should be conducted to determine precisely what knowledge and skills are needed by each category of staff. Arrangements should be made for the delivery of suitable, applied, in-service courses targeted to the most essential technical, professional, and managerial topics. Needed courses or course modules can be arranged through external educational institutions or sector training centers; or, they can be developed and offered internally by an in-house training department. Specialized training opportunities in managerial and professional or technical areas related to road maintenance often do not exist in highway agencies in sub-Saharan countries. Therefore, special initiatives may be needed to make the necessary arrangements with an appropriate educational or training institution. In undertaking such initiatives, it is important to remember that this kind of training should not be academic. It should be practical, job-related, and focused on essential skills.

CONCLUSION

The principal policy issues affecting the long-standing and serious problem of inadequate road maintenance in sub-Saharan Africa have been addressed. The single most important factor for restoration of road networks over the next decade is to obtain government commitment to road maintenance. There have been many cases in which the lack of government support has meant the failure of maintenance programs, despite the availability of funds. Attempting to set up a functioning road maintenance system is unlike building a new road, which is normally handed over in a serviceable condition on project completion. In a road maintenance project, mainly an organization and a set of patterns of behavior (systems and procedures) are put in place, all of which can disappear within a short time if there is not the will and commitment to support them. Hence, external aid for road maintenance cannot achieve anything permanent if the governments of the countries concerned are not quite convinced of the need for better road maintenance.

The first step in achieving serviceable road networks in sub-Saharan Africa will be to mobilize public and government support for concentrating scarce resources on properly planned road maintenance, rather than new construction and improvement. When this philosophy takes hold, procedures need to be developed to decide which roads are most in need of early attention, which roads can be left for later rehabilitation, and which roads can be relegated to a minimal level of maintenance. While a suitable program is set up, the best way of executing the program must be decided, taking maximum advantage of the possibilities of contract maintenance and use of increased incentives. Resource constraints in some countries and lack of interest in others mean that it is unlikely that more than a part of the backlog will be restored by the end of the century. Although some countries will be able to restore all priority roads, others will have considerable difficulty in keeping their present good and fair conditions roads in a reasonable state.

From the viewpoint of the aid agencies and their own resource constraints, it will be important that road maintenance aid is directed first to those countries that have shown commitment to better maintenance by reducing the proportion of roads expenditure spent on new works and setting up rational maintenance planning systems. It is, however, unlikely that roads aid to sub-Saharan Africa countries can be realigned completely into the necessary priority for maintenance in less than 3 years, given the scale of adjustment needed. If there is real commitment on the part of sub-Saharan Africa governments, aid donors in sub-Saharan Africa in general, and the World Bank in particular, should be planning for an increase of up to 50 percent in their financial assistance for roads in sub-Saharan Africa, with the great majority devoted to road maintenance and rehabilitation projects. The onus of reform, however, rests squarely with the sub-Saharan countries as donor assistance, and aid programs cannot substitute for vital and often difficult policy decisions needed to restore the essential road network to serviceable condition by the end of the century, and more important, to implement institutional reforms and procedural changes for cost-effective use of increasingly limited resources for the road sector.

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Secondary and Feeder Road Development Program

NELSON KUDENGA AND ANDERS BUHRMAN

Research aspects of the Secondary and Feeder Road Development Program in Zimbabwe are described. The program includes construction, upgrading, and maintenance of approximately 230 km of low-volume roads. The main research goals are (a) to determine when it may be feasible to seal a gravel road, (b) to assess the feasibility of an improved gravel wearing course, (c) to evaluate whether unsoaked CBRs could be used as design CBRs, and (d) to provide recommendations on new design, construction, and maintenance methods for low-volume roads in Zimbabwe.

The Department of State Roads (DSR) within the Ministry of Transport (MoT) is the road authority in Zimbabwe. DSR delegates some of its power to rural district councils, town councils, and municipalities. The total road network in the country is over 100 000 km. DSR is in charge of construction and maintenance on all main roads and some secondary and feeder roads. These roads have been constructed to a high technical standard, and the road network is considered one of the best in Africa.

Soon after gaining independence in 1980, the government of Zimbabwe decided to open up some previously undeveloped communal areas. The amount of traffic in these areas is low (usually less than 100 vpd), but it is increasing as agricultural output increases. Roads in these areas could not all be built using the traditional high standard for blacktop roads. Therefore, the government decided to look for alternative economical ways of building and maintaining low-volume roads in communal rural areas using methods and materials suitable to the Zimbabwean environment.

In 1986, an agreement was reached between the governments of Zimbabwe and Sweden to implement the Secondary and Feeder Road Development Program (SFRDP). This project was proposed by the Swedish National Road Consulting AB (SweRoad). The study program was presented in 1987, and the design reports for the roads to be constructed were finalized during 1988 and 1989. The construction and upgrading program started in 1989, two roads will be completed in 1990, and the remainder will be completed during 1991. One of the project roads was constructed in 1984 as a demonstration project and is included for maintenance monitoring.

PROGRAM GOALS

The four main goals of this program are to

- Initiate and introduce alternative design, construction, and maintenance concepts for low-volume roads in Zimbabwe;
- Evaluate and analyze these concepts from technical, financial, and economic points of view and arrive at recommendations regarding designs, construction methods, and maintenance strategies suitable for similar roads in Zimbabwe;
- Improve or upgrade some existing roads and tracks to higher standards; and
- Impart technical knowledge to local Zimbabweans on the design, construction, maintenance, and research methods for low-volume roads.

It is hoped that the research will answer the following two questions:

1. Is it feasible to increase the quality of the current gravel wearing course (GWC)?
2. At what traffic volumes would it be financially or economically feasible to upgrade existing earth and gravel roads using the project concept?

PROJECT ROADS

Five project roads located in five provinces servicing communal areas were chosen for the program. These roads are in different climatic, topographical, and geological environments. New road construction is envisaged for some of the roads. Others will be upgraded to a better standard. Descriptions of the roads are presented in Tables 1 and 2.

Originally it was proposed that the project roads include one or two projects constructed by labor-intensive methods to allow comparison of total financial costs, economic effects, technical standards, and benefits. This part of the project has been delayed but may be included in the final evaluation.

The design, construction, and maintenance of these roads differ to some extent from the standard approach in Zimbabwe, and the actual trials will be carried out on a number of test sections along the roads.

PROJECT ORGANIZATION

Because the project contains a large research component, MoT requested and received assistance not only from SweRoad but also from the Swedish Road and Traffic Research Institute (VTI) and the Transport and Road Research Laboratory

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TABLE 1 SFRDP PROJECT ROADS—CHARACTERISTICS

Road	Length (km)	Type	Traffic (vpd)	Remarks
St Joseph-Maphisa km 42 + 500–89 + 100	46	Gravel (completed)	60–70	Upgrading including 10-km seal
Nyika-Zaka km 0 + 000–20 + 000	20	Surfaced	200–250	New construction
Karoi-Binga km 0 + 000–38 + 000	38	Surfaced	110–250	New construction/improvements
km 38 + 000–80 + 000	42	Gravel (completed)	10–50	Upgrading/improvements
Headlands-Mayo	11 + 53	Surfaced/gravel (11 km completed)	100	8 km surfaced; 2 km gravel; maintenance on rest
Kamativi-Binga	20	Surfaced (completed 1984)	40	Monitoring of deterioration; maintenance

TABLE 2 SFRDP PROJECT ROADS—ENVIRONMENT

Road	Rainfall (mm)	Soil Type	Economic Activity
St Joseph-Maphisa	400–500	Alkali sand with clay pockets	Agriculture; irrigation schemes; cattle ranching
Nyika-Zaka	600–800	Gneissic sand with clay and sandy clay	Agriculture on subsistence level
Karoi-Binga	800–900	Gneissic sand with clay and sandy clay	Agriculture; fishing
Headlands-Mayo	1000–1200	Granitic and gneissic sands and sandy clay	Agriculture; livestock
Kamativi-Binga	600–800	Expansive shale soils	Fishing; tourism; agriculture on subsistence level

(TRRL) in the United Kingdom. To facilitate cooperation between the supporting agencies and MoT staff and to ensure overall control by MoT, project monitoring was delegated to a special unit within MoT—the Project Monitoring and Evaluation unit (PME). PME is staffed by engineers and researchers from MoT, the Swedish National Road Administration, and VTI, with short-term input from TRRL.

GENERAL APPROACH AND IMPLEMENTATION

Design

The project roads were designed using the existing roads as much as possible but aiming at design speeds of 80 or 60 km/hr depending on road type and terrain. Deviations were allowed where strict adherence to these design criteria could not be justified economically or considering traffic safety aspects. The survey and design were carried out using modern instruments and computers and with a minimum of the standard MoT referencing. The design program used was GEOSECMA (*1*,*2*).

The pavement design for the sealed roads was based on MoT recommendations (*3*), substituting Texas triaxial tests with California bearing ratio (CBR) values as the main design criteria and reducing the number of pavement layers to subbase and base only. There was a possibility that design CBRs taken at equilibrium or optimum moisture content (OMC) could be used instead of using soaked CBRs, not only at designated test sections but also at other places where the conditions were favorable. However, the design consultant could not take advantage of this opportunity because of a lack of reliable data on OMCs and because of unexplained variations in the unsoaked test results achieved through DCP tests. Both of these aspects have been covered in the research program and the PME design for improvements. The latter includes the use of existing GWC as subbase (Headlands-Mayo) or base (Kamativi-Binga), assuming unsoaked conditions (*4*,*5*).

The surface of the sealed roads is being constructed with a graded gravel aggregate known as Otta-seal or YIG (see Figure 1). This surface is used mainly in Scandinavia but also on low-volume roads in Botswana and Kenya (*6*).

The Otta-seal will provide a dense, flexible cover that, depending on the bitumen used, could have self-sealing abilities. Previous experiences in Scandinavia and Botswana indicate that the best results are achieved by using a double seal, with the second seal applied 2 to 15 months later.

The pavement design for the gravel roads is new to Zimbabwe and was based on the specifications presented in Table 3.

The GWC used on the project roads should fall as close as possible to the inner envelope shown in Figure 2, have a PI of 10 to 15, and preferably have a clay content of at least 5 percent.

The project GWC is thinner than the standard layer (10 cm versus 15 cm) but of considerably higher quality than the normal MoT standard, by which gravel roads can have a variable surface quality.

Construction

As mentioned, construction is being carried out using MoT normal construction and improvement units. Assistance is provided for planning and follow-up as well as for direct supervision of the test sections (e.g., conditional approvals of compaction are not accepted). Detailed referencing of the road is carried out during the construction stage.

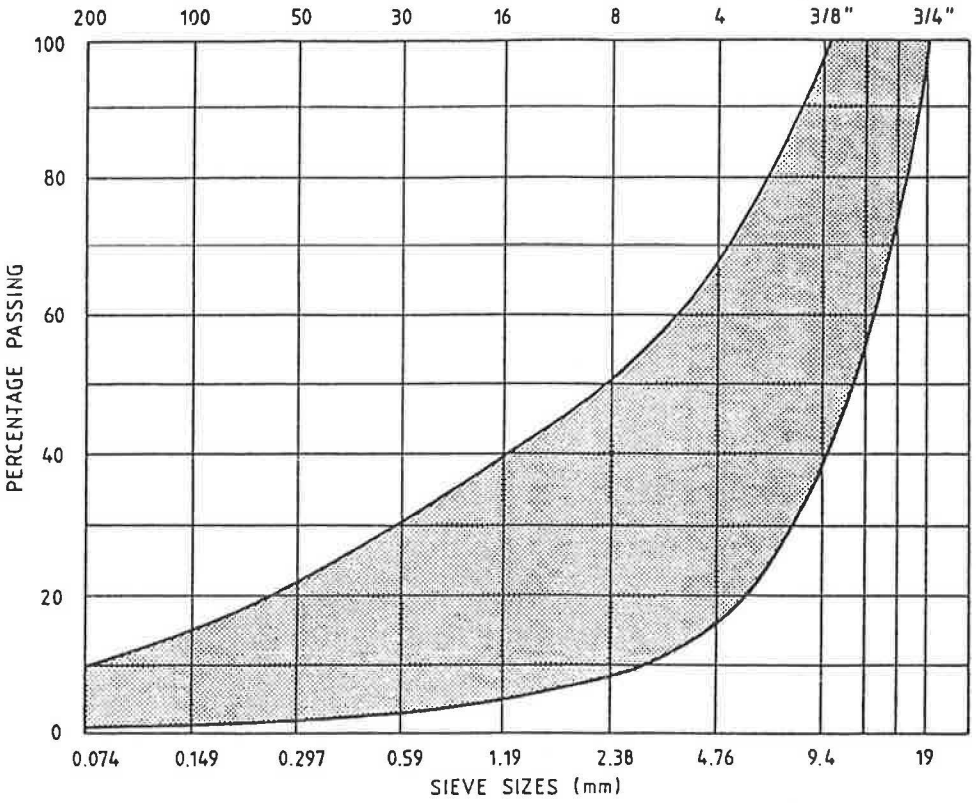

FIGURE 1 Grading envelope for specification of Otta-seal.

TABLE 3 PAVEMENT DESIGN FOR GRAVEL ROADS

Subgrade Condition	Thickness of Wearing Course (mm)	Thickness of Selected Fill or Subbase (mm)	Total Thickness (mm)
>SG 9	100	0	100
SG 5	100	125	225
SG 3	100	200	300
<SG 3	100	300	400

NOTE: Pavement standard design = 0.05 M. SG 5 = subgrade with 5 percent < CBR < 9 percent. The subgrade condition will be assessed from soaked CBR tests carried out at a compacted density of 90 percent modified AASHTO and at the modified AASHTO OMC. The subgrade will be compacted to a density of 90 percent or greater of modified AASHTO maximum dry density. The selected fill or subbase will be compacted to a density of 93 percent or greater of modified AASHTO maximum dry density. The wearing course will be compacted to a density of 95 percent or greater of modified AASHTO maximum dry density. The moisture content used for compaction of the wearing course and the selected fill or subbase will be within 2 percent of the OMC for the modified AASHTO compaction. This specification is only applicable for the test sections.

The maximum stone size for the GWC was set at 20 mm (Scandinavian specifications). Adherence to the envelope will normally require that the GWC be crushed or screened, or both. Therefore, mobile crusher/screener suitable for the conditions in Zimbabwe was procured for the project. The importance of controlling the maximum stone size has been supported by other research in the region (*7*). In addition to crushing or screening, the gravel material must be mixed with clay in some areas to meet the PI requirement.

Compared with existing MoT standard design and construction methods, construction costs for the project are about the same for the gravel roads but are lower for the bitumen roads. Available figures (excluding Nyika-Zaka) are presented in Table 4.

Maintenance

The project will include evaluation of different maintenance policies, especially for routine gravel road maintenance. The surface width (6 m) was selected on the basis of studies that used graders with mounted windrow eliminators, which increased grading productivity by 60 percent in Zimbabwe (*8*).

Another grader attachment used successfully on the improvement projects is the ditch plough, which is mounted directly on the grader blade and has proven to be very cost efficient with loose and normal materials. The windrow eliminator and ditch plough are now manufactured in Zimbabwe.

RESEARCH PROGRAM

Main Goals

To be able to provide well-founded and objective recommendations, a sizable research program was added to the project. The program was designed to allow comparisons and use of results from similar research (e.g., in Kenya, India, and Brazil). The research will be concentrated on the following issues and objectives:

- Determination of when it may be feasible to seal a gravel road,

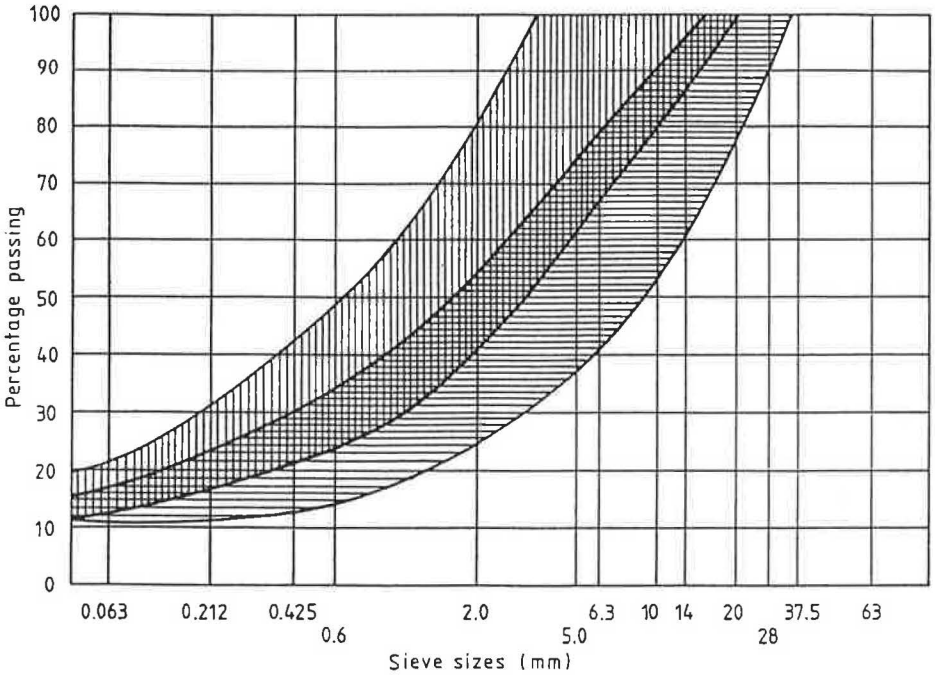

FIGURE 2 Grading envelope for gravel wearing course.

TABLE 4 CONSTRUCTION COSTS BY ROAD TYPE

Road Type	Width (m)	Construction Cost (Z$ per km)
Bitumen MoT design	6/7[a]	315 000
Bitumen, Project Design 1	6/7	210 000
Bitumen, Project Design 2	6/6	110 000
Gravel MoT	0/6	110 000
Gravel, Project Design	0/6	110 000

[a]6-m seal on 7-m formation.
NOTE: Costs reflect 1991 prices. For bitumen roads, Project Design 1 = standard design done by SweRoad and Project Design 2 = design by PME on improvements, using unsoaked design CBR (normally one step lower in the design table) and without deviations from existing road alignment.

- Determination of the feasibility of an improved GWC,
- Evaluation of whether unsoaked CBRs could be used as design CBRs for low-volume sealed roads in certain parts of Zimbabwe,
- Determination of the feasibility of using DCP tests or the Clegg hammer to obtain design CBR values as a supplement to normal methods,
- Evaluation of new construction and maintenance methods, and
- Development of and recommendations for design charts for low-volume roads in Zimbabwe.

Variables

The variables to be considered in the research program are as follows:

Gravel Roads

- Improved-quality wearing course,
- Reduced thickness of wearing course with improved-quality material,
- Different maintenance strategies,
- Different traffic levels (depending on the siting of the test sections),
- Different climate (depending on the siting of the test sections), and
- Use of dust palliatives (i.e., surface binders).

Sealed Roads

- Different bituminous seals including priming and surface treatment,
- Different pavement thickness designs,
- Different quality materials for road base,
- Different shoulder treatment,
- Different traffic levels (depending on the siting of the test sections), and
- Different climate (depending on the siting of the test sections).

Table 5 presents details on variables and measurement methods.

Test Sections

The program includes 26 test sections, of which 9 are located on gravel road sections, 14 on designed bitumen sections, and 3 on improved sealed sections. The gravel road test sections (1 km each) are further subdivided into three subsections with variable maintenance levels. Most of the test sections are straight and level with uniform subgrades and include control sections of standard MoT design. An example of the test sections selected for one of the project roads, Headlands-Mayo, is shown in Figure 3.

TABLE 5 MONITORING OF ROAD SECTIONS

Variable	Frequency	Method/Equipment	Applicable Road Type
Traffic composition	Twice/year: 7 days, 6 am to 6 pm	Manual (MoT)	G;S[a]
Traffic volume	7 days, 6 am to 6 pm	Manual plus counters (MoT)	G;S
Traffic loading	7 days, 6 am to 6 pm	Loadometers (MoT) or weighbridges	G;S
Traffic speed	Before and after maintenance	Radar pistol and clocks	G;S
Pavement strength and condition	Beginning and end of wet season and middle of dry season (G); twice a year (S)	Instructions for measurements separately issued	G;S
In situ CBR	Beginning and end of wet season and middle of dry season (G); twice a year (S)	DCP	G;S
Density	Beginning and end of wet season and middle of dry season (G); twice a year (S)	Troxler or CPN Strata Gauge	G;S
Moisture content	Beginning and end of wet season and middle of dry season (G); twice a year (S)	Troxler or CPN Strata Gauge	G;S
Impact value	Beginning and end of wet season and middle of dry season (G); twice a year (S)	Clegg Impact Tester	G
Indicator tests	Immediately after construction, after 1 year, then yearly	Samples for grading, PI, standard tests (in transition zones between test sections)	G
Deflection	Once before construction, then twice a year (end of dry season and end of wet season)	FWD	G;S
Deformation (G)	Before and after maintenance plus monthly	Instructions in appendix	G
Corrugation	Before and after maintenance plus monthly	Assessment, photos, and sketch	G
Potholes	Before and after maintenance plus monthly	Assessment, photos, and sketch	G
Dusting	Before and after maintenance plus monthly	Assessment	G
Looseness	Before and after maintenance plus monthly	Assessment	G
Rutting (maximum rut depth)	Before and after maintenance plus monthly	2-m straightedge over wheel tracks in several cross sections along test sections	G
Deformation (S)	Twice/year	Instructions in separate report	S
Cracking	Twice/year	A frame (1 m^2) at each deflection chainage point.	
Rutting (maximum rut depth)	Twice/year	2-m straightedge over wheel tracks in several cross sections along test sections.	
Surface texture	Twice/year	Preferably the Mini Texture Meter (TRRL); otherwise, UCON sand patch test in each wheel track and one in centerline along each test section	
Roughness	Before and after maintenance plus monthly	Bump Integrator and Abay beam	G
Roughness	Twice first year, then reassessed	Bump Integrator and Abay beam	S
Gravel loss	• Sections with nil maintenance: 3 times/ year (4-month interval)	Leveling in cross sections with optical instruments. Cross-sections initially have	G

TABLE 5 *(continued on next page)*

TABLE 5 *(continued)*

Variable	Frequency	Method/Equipment	Applicable Road Type
	• Sections with one heavy grading: before and after treatment • Sections with two heavy gradings: before and after each treatment	c/c 5 m for comparison. A change to c/c 50 m over whole length of test section might be proposed after some experience.	
Permeability in pavement	Once/year	Not yet specified	S
Groundwater	If necessary	Monthly recording of GWL in pipes in open ground adjacent to road	G;S
Rainfall	Daily during wet season	Rain gauge installed at some official buildings (e.g., a school close to project roads)	G;S

[a]G = gravel roads; S = surfaced roads.

NOTE: Visual inspections will also be carried out two or three times per year to subjectively assess the road conditions according to standardized inspection forms used in the 1985 and 1989 Zimbabwe Road Condition Surveys.

TEST SECTIONS ON ROAD 239 HEADLANDS - MAYO (HSMO)

GRAVEL SURFACED KM 9 -11

TEST SECTION NO	1			2
CHAINAGE	KM 9 +000- 10 +000			10+000 - 11+000
LENGTH/WIDTH	1000M/6M			1000M/6M
SURFACING	WC 100 MM ACC. TO PROJ.			WC 100 MM ACC. TO MOT
SUBBASE/SELECTED FILL	175 MM			175 MM
SUBGRADE	SG 5			SG 5
MAINTENANCE/ LENGTH	NIL 300 M	1 GRADING 300 M	2 GRADINGS 300 M	ACC. TO SECTION 1

SEALED SURFACED KM 0 - 9

TEST SECTION NO	1
CHAINAGE	KM 5 +800 - 6 +300
LENGTH/ WIDTH	500M/6M
SURFACING	SINGLE OTTA SEAL
BASE	150 MM
SUBBASE	0
SUBGRADE	SG 9

FIGURE 3 Test sections on project road.

Detailed preinvestigations have been performed on all test sections. The preinvestigations included soil classifications, layer thicknesses, in situ and laboratory CBR tests, moisture contents, PI values, Clegg hammer and DCP values, falling weight deflectometer (FWD) data, and roughness data over time.

Time Schedule and Resources

The field measurements will require at least three field assistants supported by two full-time researchers in the PME during the first phase, which should end in mid-1992. At that time, research on the gravel roads should be complete and a preliminary report on the sealed roads should be made available.

The final report on the sealed roads cannot be expected until at least 5 years after the roads have been constructed, and the final answers on some questions may only be available after 8 to 10 years or when a complete overlay is required.

Intermediate Conclusions

From the preinvestigations and subsequent monitoring of the test sections, the following preliminary conclusions have been made:

- The lowest in situ CBR values (down to 600 mm) generally exceed the laboratory soaked CBR values, indicating that a pavement design that is based on the latter would be very conservative. The results tend to support upgrading the subgrade classification by at least one step (*9*).
- Current design methodology, which is based on a limited number of soaked CBR values and grouping of subgrade samples, is unreliable (*9*).
- A design methodology that is based on a large number of DCP values taken in March and assessed statistically would be more informative for the designer (*9*).
- A good correlation between DCP and CIV has only been established in sand. For other materials, it has not been possible to establish a general relationship (*10*).
- The performance of the Kamativi-Binga and Headlands-Mayo roads supports the view that low-cost sealed roads are more cost efficient than standard gravel roads in Zimbabwe (*4–6*).

EVALUATION

Evaluation of the alternatives will be based both on performance (i.e., assessments of standard and rate of deterioration) and on financial and economic costs. The economic costs will be calculated using both the HDM III and RTIM 2 computer models. In addition, checks will be made with the NTS (*11*). Accident costs will probably not be included in the economic calculations because their influence will likely be marginal.

The more detailed performance evaluations and individual relationships found may possibly be reported continuously, at least on the gravel road trials. Findings and results documented during 1990 and 1991 will be presented at the TRB 5th International Conference on Low-Volume Roads.

ACKNOWLEDGMENTS

This project was funded by the Swedish International Development Agency. The authors are grateful to the Zimbabwe Secretary for Transport for his permission to publish this paper.

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Economic Appraisal of Guinea-Bissau Roads Program Using HDM–III and EBM Models

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Guinea-Bissau is a small country on the West African coast. With an estimated per capita income of about U.S. $170, this country is one of the poorest in the world. The economy is characterized by a large traditional rural sector, producing primarily for subsistence and agriculture. Fisheries and forestry are the other main activities. The country currently suffers from an extremely dilapidated infrastructure, and roads are no exception. A 5-year road program has been designed in connection with a World Bank project to produce an acceptable mix of road conditions. An economic analysis of the 5-year road program was conducted using the Highway Design and Maintenance Standards Model. Rates of return and net present values of alternative improvement strategies are given both for paved and unpaved road sections. The optimum set of road sections and corresponding maintenance options are then obtained with application of the Expenditure Budgeting Model. The resulting road network will help to increase the country's level of economic activity through reduced freight and passenger transportation costs, more efficient transportation of goods to consumption centers and ports, increased mobility for the population as a whole, easier access to medical and social services, and improved road safety for vehicles and pedestrians.

Guinea-Bissau, with a land area of 36 125 km², is a small country located between Senegal and Guinea-Conakry on the West African coast. The country includes the 30 islands of the Bijagos Archipelago. The population is estimated at 900,000 inhabitants, 15 percent of whom live in the capital, Bissau, or its periphery. Population density is about 25 residents per square kilometer. With an estimated per capita income of $170, Guinea-Bissau is one of the poorest countries of the world. The rural sector employs almost 90 percent of the population, and the industrial and commercial sectors employ less than 5 percent. Local production includes rice (about 155,000 tons/year), fruits, and, more recently, cotton. Forest covers about 2.3 million hectares of land, but this area is diminishing rapidly, through timber extraction (about 15,000 tons/year) and forest fires. Mineral resources (bauxite, phosphate, and probably petroleum) are still unexplored. Principal exports are cashews, groundnut, palm kernels, and timber. With a coast 220 km long and an archipelago of many islands, there is great potential for a fishing industry, but fishing is done by foreign fleets to which fishing licenses were conceded, with limited benefit for Guinea-Bissau.

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Although in the past few years positive results have been verified in the development of some sectors of activity, as a result of the liberalization of commerce and industry, the country presently suffers from an extremely deteriorated infrastructure, and its road network is no exception.

ROAD NETWORK

The road network has a total of 2636 km, of which 544 km are paved, 308 km are laterite gravel, and 1784 km are earth roads. These figures do not include some roads that are very local and rural in nature, as well as the small paths that access small villages. Thus, the country is reasonably serviced by the existing road network and it is not considered necessary, in the short or even medium term, to extend the network. It could even be argued that the density of about 70 km of road per 1000 km² is excessive, because it is above the average for African countries. However, the physical characteristics of the territory, with numerous and deep sea inlets, a complex river system, and low elevations, must be taken into consideration. Probably a higher number of ferries could reduce the need for a relatively long road network. However, low traffic volumes in several zones would not justify the investment and operating costs of ferries. Moreover, the generally deficient management of ferries would not contribute to an optimal use of the road network.

In terms of geometric characteristics, the network is not uniform; in any one category of roads, varying widths of roadway and shoulders can be seen. On paved roads, roadway width ranges between 5.0 and 7.0 m, and that of shoulders, between 0.5 and 2.5 m. On laterite gravel roads, the roadway width ranges between 5.0 and 7.0 m. For earth roads, built without a geometric design, it is not easy to define the roadway width, because vehicles, attempting to avoid obstacles, drive over the side of the road; the earth, compacted by vehicles, can be as much as 20 to 30 m wide.

In terms of technical characteristics, the roads do not present uniform criteria, either in the construction of the pavements, or the materials from which they are built. For example, there exist paved roads with a traffic volume of 10 or 20 vehicles per day (vpd), whereas other roads have a volume of 100 to 150 vpd without even a laterite gravel surface of adequate gradation. It is believed that this condition, inherited from colonial times, stems more from the military strat-

egies of the country's protracted war for independence, than any other technical reason.

The oldest paved roads are constructed with a base of varying thickness, comprised of laterite soil and a single or double bituminous surface treatment, using laterite gravel as aggregate, which have sustained traffic for up to 20 years. On more recently built or reconstructed paved roads, dolorite aggregate is used for bituminous surface course, instead of laterite. With two or three exceptions, earth and laterite roads are in a deplorable condition. Basic characteristics of laterites found in Guinea-Bissau are presented in Table 1.

The drainage system on Guinea-Bissau roads is quite deficient, responsible in large part for many of the network's problems and accidents. Annual rainfall is about 2000 mm/year, concentrated in 3 months (between June and September), with as much as 800 mm in only 1 month.

Based on an inventory taken nationally in 1988, and taking into account some recent repairs, as well as projected further deterioration, the current condition of the road network is estimated as presented in Table 2. Besides these roads of the classified network, there are scores of kilometers of rural roads that provide precarious access to small villages or production zones.

In order to give continuity to the emergency interventions already completed or underway for improving the road network, preparing a road rehabilitation and maintenance program that would include the most recommended interventions, on the highest priority road sections, with the highest rate of return on investments made was considered necessary.

TABLE 1 MAIN CHARACTERISTICS OF LATERITE SOILS

Samples	106/89	48/89
Classification	A-1-a	A-2-4
Sieve Analyze (% passing)		
Sieve 1 1/2"	100	-
" 1"	92	100
" 3/4"	78	93
" 1/2"	71	74
" 3/8"	66	64
" No.4	46	47
" No.10	33	43
" No.20	27	42
" No.40	23	41
" No.80	18	37
" No.200	14	28
Plasticity		
Liquid limit %	32	29
Plasticity limit %	26	21
Plasticity index %	6	8
CBR Testing	75	53

OBJECTIVES OF AN IMPROVEMENT PROGRAM

The set of interventions to be included in a proposed 5-year road rehabilitation and maintenance program should reverse the negative trend of the continuing, accelerated deterioration of the roads in general, and initiate an irreversible process of road network improvement that would

- Reduce freight and passenger transportation costs;
- Result in more efficient transportation of goods to consumption centers and ports;
- Increase mobility of the population and improve access to medical and social services; and
- Improve road safety for vehicles and pedestrians.

METHODOLOGY

Given the country's limited resources for public expenditures and the necessity for foreign exchange to undertake road works, it was necessary to seek financing from international and bilateral institutions for the work to be undertaken on the road network. The road improvement program should therefore be based on an equilibrium between the available financial means and the interventions with the highest economic return. Seeking to use a methodology in program preparation that would optimize the investment made, the Highway Design and Maintenance Standards Model (HDM–III) and the Expenditure Budgeting Model (EBM) (*1*,*2*) were selected. The procedures adopted are described in the following sections.

DATA BASE

On the basis of an inventory undertaken of all of the country's road network, the available traffic counts, and the elements of analytical accounting for road maintenance works, a data base could be developed for use with the adopted methodology.

Traffic volume is quite low throughout the country, which is not surprising because in early 1989 there existed only 7,500

TABLE 2 ROAD NETWORK CONDITION

Type of Roads	Total	Good		Fair		Poor	
	km	km	%	km	%	km	%
Paved Roads	544	268	49	94	17	182	34
Laterite Gravel Roads	308	55	18	133	43	120	39
Earth Roads	1784	75	4	161	9	1548	87
TOTALS	2636	398		388		1850	

vehicles in the whole country, a large percentage of which was, and still is, concentrated in the zone of Bissau.

In 1986, a traffic census unit was created that has, since then, been taking counts in most of the posts throughout the country. Thus, a real sample of traffic volume is available, because the counts are taken for a full week at each post, with separate counts for every hour between 07:00 and 20:00. During one of the work days, nocturnal traffic volume is also counted, between 20:00 and 07:00. The census is also differentiated by nine different types of vehicles. Because the counts are taken at various times of the year and every day of the week, it is possible to evaluate the seasonal and daily variations throughout the country.

If the only access road to Bissau (where, close to the airport, 8 km from Bissau, traffic volume reaches 5,000 vpd) is excluded, the traffic volumes on the principal roads vary between 200 and 650 vpd and on the secondary roads, between 50 to 200 vpd. The local access roads have a traffic volume between 20 and 80 vpd, and the rural road's traffic volume varies between 5 to 20 vpd. The breakdown of traffic by types is clearly characterized in the different zones of the country. As was already mentioned, the largest number of motorized vehicles is found in the zone of the capital; in the northern and eastern part of the country, bicycles predominate, and in the latter zone, there are vehicles pulled by animal traction also. Of motorized vehicles, the breakdown between light and heavy vehicles varies considerably, especially between the zone of the capital and the provinces. Although in the capital the percentage of heavy vehicles is on the order of 9 to 15 percent of the total volume, this percentage is much higher in the rural area, reaching, at some periods of the year, 30 to 40 percent. The importance of this fact and its impact on roadway conditions should be noted because most heavy vehicles seldom carry a full cargo; often the cargo per journey is substantially less than the maximum permitted by law. On the other hand, a small number of heavy vehicles carry cargo that substantially exceeds the legal cargo allowance.

For the technical and economic evaluation of the network, besides traffic volumes the composition of the fleet of vehicles by categories is important. The totality of the vehicles of Guinea-Bissau was analyzed to determine the fleet of vehicles that best characterizes it. With this analysis, and considering the almost total absence of buses for public transportation, the fleet is composed of five types of vehicles:

LIG	Cars (passenger, or mixed-use)
UTI	Pick-up trucks
CAML	Light trucks for freight
CAMM	Medium trucks for freight
CAMP	Heavy trucks for freight

Even in the absence of relevant statistical data before 1986, it can be verified that the volume of traffic has increased or decreased as a function of the economic situation of the country. Periods in which fuel and spare parts were scarce resulted in extremely low traffic volumes, in contrast to the recent, relatively substantial increase in the number of vehicles, and consequently traffic volume, a result of the liberalization of commerce. To a large degree, this increase is a result of the importation of used cars from Europe, which are not always in adequate maintenance condition. From the traffic counts, it is possible to note the considerable increase in traffic volume in the last 2 years, and in some posts, annual increases of up to 100 percent can be noted, possible only because of the low total volumes being considered.

The assumed traffic values are a result of the most recent counts, in most cases taken between 1987 and 1988. The adopted rate of increase varies according to the road being studied; the period between 1990 and 1992 has a higher value and the period after 1992, a lower value. The two rates, which in most cases are 8 and 6 percent, respectively, are not representative of the increases currently verified, which are superior to these; the lower values were adopted for the sake of prudence. For the roads that serve the new river ports with a current very low volume of traffic, an 18 percent growth rate for the first period and 16 percent for the second period is assumed, given the development pole that they serve.

ROAD NETWORK CONDITION

The road network condition was already analyzed in its general context. Now it is necessary to determine the condition levels of each section on the basis of the inventory undertaken and relate this condition with a roughness level to be used in the calculations. No means were available to measure road roughness in Guinea-Bissau. Therefore, for preparing this program, six levels of road conditions were associated through visual inspection of the roads with six levels of the International Roughness Index IRI (*3*), as indicated in Table 3. Given this relationship, the condition of each road section will be referred to in terms of its degree of roughness.

AVAILABLE SOILS

There is not a great variety of soil types in Guinea-Bissau; the laterites, silts or silty clays, red or yellow sands or clays predominate, with some outcroppings of dolorites in the center of the country.

The laterites vary in gradation, and in the southeast of the country, there are numerous, significant rock outcroppings (caprocks). Up to now, the use of laterites in road construction has been limited to the use of the laterite as it is extracted, and selected according to their gradation. However, it will be possible to crush laterite boulders to get better materials for roads or runways that require better quality bases and subbases. Because of the predominance of these laterite soils in this region of Africa, and given its excellent quality for road construction, either as a base in paved roads, or as a surfacing in unpaved roads, and even as an aggregate in bituminous pavement, it would not be difficult for Guinea-Bissau to have a good road network. It would only be necessary to rehabilitate the existing network and maintain it. The laterite in itself ensures a good-quality roadway (*4*,*5*).

UNIT COSTS

All the costs necessary for the technical and economic appraisal of the network in all its diverse conditions will be analyzed in the following paragraphs. Given the continued devaluation of the Guinean currency (Peso), all values are expressed in U.S. dollars of January 1989.

TABLE 3 RELATION BETWEEN ROAD CONDITION AND ROUGHNESS LEVEL

A. PAVED ROADS

Level 1:	Recently paved roads in excellent condition. IRI = 2.0 m/km
Level 2:	Paved road which provides good riding comfort and maintains its profile. May present slight distortions which do not, however, affect surface drainage. IRI = 3.5 m/km
Level 3:	Paved road which presents a noticeable distorted profile. However, these distortions affect less than half of the pavement surface and do not impose travel speed reduction. IRI = 6.0 m/km
Level 4:	Paved road whose surface is damage and presents numerous incidents of pavement failures: distortions, ruts, depressions, cracks, etc. Failures remain light and only impose a slight reduction on travel speed. IRI = 8.0 m/km
Level 5:	Paved road whose surface is heavily deteriorated and presents numerous incidents of major pavement failure: distortions,pot holes, cracks, etc. Failures are severe enough to impose a substantial reduction on travel speed. IRI = 10.0 m/km
Level 6:	Paved road whose surface is completely destroyed. Pavement failures are such that vehicles can only travel with difficulty and in so doing may incur tire, suspension or even frame damage. IRI = 12.0 m/km

B. UNPAVED ROADS

Level 1:	Recently constructed or reconstructed gravel road having excellent transverse profile, riding qualities and good drainage. IRI = 3.0 m/km
Level 2:	Gravel road, structurally sufficient, may present small irregularities, such as light corrugations or slight wear along the wheel tracks, which could be readily corrected with a light cut by a conventional road grader. IRI = 4.0 m/km
Level 3:	Gravel road where surface irregularities constrain the comfortable travel speed to a speed below that dictated by the alignment. Correction would require a heavy grader cut or a re-graveling. IRI = 6.5 m/km
Level 4:	Gravel road where a significant reduction in travel speed is required to attain safe and comfortable driving conditions. This level would apply to sections where the subgrade is visible. IRI = 9.0 m/km
Level 5:	Gravel road which has failed to the extent that a detour is in operation. IRI = 12.0 m/km
Level 6:	Earth road without any maintenance. The condition level varies with nature of terrain and soils. IRI = 15.0 m/km

The following unit prices for different maintenance tasks, which include labor, equipment, materials, and overhead costs, were adopted:

Task	*Unit Price (U.S.$)*
Grading	125/km
Spot regraveling	13/m³
Patching	9/m²
Resurfacing (paved roads)	11/m³
Resurfacing (unpaved roads)	2.8/m²
Routine maintenance (paved roads)	260/km-year
Routine maintenance (unpaved roads)	380/km-year

Table 4 presents unit costs for each type of vehicle considered, including costs of new vehicle, tires, fuel, and labor for maintenance and operation.

TRANSPORT POLICY SCENARIOS

Because of the physical characteristics of Guinea-Bissau, with an archipelago of numerous islands and deep sea inlets, much research has been done to define the transportation policy that should be implemented. Some have recommended that roadway transportation should be strengthened, whereas others have advocated that river and maritime transportation should take precedence. The national volumes of production are extremely low, volumes to be transported are equally very low and the territory is quite small. Therefore, for both modes of transportation, problems exist and must be taken into consideration. For river transport, the mode highly recommended for great volumes of freight or large number of passengers, there is the problem of short transport distances, and the inability to deliver door-to-door. On the other hand, roadway transportation requires more expensive infrastructure but has the advantage of picking up goods at the production site and delivering them to their final destination.

Recently, the *Strategy for a Transport Policy* (*6*) established that the combination of the two transport systems should be improved, for a gradual, more rapid national development. However, vehicle operating costs should be lowered because, presently, given the condition of the roads, these costs can reach U.S.$2.54/km for a truck. Given this context, the decision was taken to invest in the rehabilitation of the road network as a way to promote transportation throughout the country, with acceptable operating costs.

SELECTION OF REPRESENTATIVE ROAD SECTIONS

The road network was divided into homogeneous road sections with identical characteristics, with more or less the same degree of deterioration and about the same traffic level. On the basis of this criterion, 22 road sections were selected as representative of the whole road network.

Table 5 presents the selected sections of the road network broken down by paved or unpaved roads. The table gives the length and identification code, as used in the analyses, for each road section (or link).

REHABILITATION AND MAINTENANCE STRATEGIES

Two levels of rehabilitation were considered in the analyses, both for paved and unpaved roads, described as follows:

RL01

Light rehabilitation of paved roads consisting of the repair of the existing pavement, improving the shoulders, constructing or correcting culverts or other drainage elements, and, finally, applying a bituminous surface course, over the repaired pavement. This rehabilitation would elevate a Level 4 (IRI = 8.0 m/km) road to a Level 2 (IRI = 3.5 m/km) condition (see Table 3).

RP01

Heavy rehabilitation of paved roads consisting in the improvement of the subbase with a layer of selected laterite gravel, with a thickness of 10, 15, or 20 cm; improving the shoulders; constructing or correcting culverts and other drainage elements; and, finally, applying a bituminous surface course, over a properly prepared base. This rehabilitation would elevate a Level 5 or 6 road to a Level 2 condition.

RL02

Light rehabilitation of unpaved roads consisting of the reinforcement of the laterite roadway layer up to 15 cm of thickness with a final grading and mechanical compaction. Drainage elements would be corrected or constructed to ensure good drainage conditions. This rehabilitation would elevate a Level 4 road to a Level 2 (IRI = 4.0 m/km) condition.

RP02

Heavy rehabilitation of unpaved roads, consisting of upgrading an earth road to a laterite gravel road with at least 15-cm

TABLE 4 VEHICLE UNIT COSTS IN U.S. DOLLARS

TYPE OF VEHICLE	COST	TIRES	FUEL	LABOR	CREW
LIG	14,000	150	.55	.50	.00
UTI	16,000	180	.20	.50	.30
CAML	31,000	280	.20	.50	.30
CAMM	56,000	400	.20	.70	.50
CAMP	95,000	450	.20	.70	.50

TABLE 5 ROAD SECTIONS AND INPUT DATA SUMMARY

ROAD No.	Section	LENGTH m	RUN CALC. REF.	ALTERNATIVES Base	Rehal	Reha2	TRAFFIC vpd	%Grow.	IRI (m/km) Befo.	After	REHAB. YEAR
PAVED ROADS											
N1	Biss-Jugud	47	N100	ALT0	ALT1		669	8-6	10.9	3.7	1990/91
N2	Safi-J.Lan	8	N200	ALT0	ALT1		323	8-6	10.7	3.7	1991
N1	Jugu-Bamba	62	N1A0	ALT0	ALT1		274	8-6	8.6	3.7	1992/93
R9	Bamba-Xime	11	R900	ALT0	ALT1		45	18-12	11.5	4.7	1993
R7	Catio-Impu	18	R700	ALT0	ALT1		53	18-12	10.5	4.7	1993
L35	Jembe-Cadi	11	L350	ALT0	ALT1		42	18-12	11.5	4.7	1993
R3	Manso-Biss	25	R300	ALT0	ALT1		133	8-6	11.5	4.7	1992
UNPAVED ROADS											
N1	Caium-Buru	16	N101	ALT0	ALT1	ALT2	100	8-6	12.5	4.8/3.6	1991/92
R4	N1-Contubo	19	R401	ALT0	ALT1	ALT2	148	8-6	12.5	3.9/3.6	1991
L24	N1-Galomar	24	L240	ALT0	ALT1	ALT2	125	8-6	9.7	3.9/3.6	1991
R4	Cont-Front	40	R402	ALT0	ALT1		71	8-6	12.6	3.6	1991/92
R8	N.Sin-S.Jo	13	R800	ALT0	ALT1		36	18-12	12.6	3.8	1991
N2	Mamp-Guile	30	N202	ALT0	ALT1		47	8-6	12.6	3.9	1992
L18	N1-Gambiel	37	L180	ALT0	ALT1		65	8-6	9.4	4.0	1993
L24	Gal-Dulomb	20	L24A	ALT0	ALT1		64	8-6	9.5	4.0	1993
R7	Buba-Catio	52	R7A0	ALT0	ALT1		55	8-6	11.1	3.9	1992/93
N2	Enxud-Buba	73	N201	ALT0	ALT1		73	8-6	9.5	4.0	1993/94
R1	Man-Bafata	64	R101	ALT0	ALT1		49	8-6	10.1	3.9	1993/94
N3	Ingore-Far	66	N301	ALT0	ALT1		37	8-6	14.2	3.9	1993/94
R1	Binar-Biss	24	R1A0	ALT0	ALT1		37	8-6	10.9	3.9	1994
L30	Ban-Empada	13	L300	ALT0	ALT1		35	8-6	11.8	4.0	1992
L16	Bafa-Gamam	13	L160	ALT0	ALT1		34	8-6	11.5	4.2	1993

thickness of laterite layer. The road would be elevated through embankments. Culverts and other drainage elements would be built to drain water away from the roadbed. The roadway would be constructed of selected laterite gravel, correctly graded and compacted. In some sections, the road could be newly aligned if the existing roadway were below the grade of the adjacent land, a result of numerous regradings without resurfacing or to avoid its alignment through a small village. This rehabilitation would elevate a Level 5 or 6 road to a Level 2 (IRI = 4.0 m/km) road condition.

MAINTENANCE

Two types of intervention were considered for paved roads and two for unpaved roads, referring to them as follows with their identification code as it appears in the calculations:

MAN3

This level of maintenance applied to paved roads consists of the minimum interventions necessary to maintain a road usable, even if it is possible to verify an accelerated deterioration as a result of a weak base. This level consists of only two tasks: repair potholes to 20 percent and small routine maintenance tasks.

MAN4

This level of maintenance also applied to paved roads consists of all the tasks that should be included in an adequate continuous maintenance program to maintain the road in good condition; the deterioration evident would correspond to the normal wear as a result of vehicular circulation. This level includes the following tasks: repair potholes to 100 percent, seal cracks to 100 percent, and complete routine maintenance.

RESE

This code identifies the periodic maintenance tasks to be applied on paved roads approximately every 7 years, in accordance with the level of traffic and consists of pavement repair, resurfacing with a new bituminous surface course, and reinforcement of the shoulders.

MAN1

This level of maintenance applied to unpaved roads includes only the minimum interventions necessary to allow circulation, even with some degree of difficulty in some periods of the year. Basically, the tasks consist of replacing the material eroded up to 20 percent and regrading every 1 or 2 years in accordance with traffic.

MAN2

This level of maintenance also applied to unpaved roads consists of all the tasks that should be included in an adequate continuous maintenance program to maintain the road in good condition; the deterioration evident would correspond to the normal wear as a result of vehicular circulation. This level includes the following tasks: replacing the lost material up to 70 percent, regrading periodically every 120 to 360 days in accordance with traffic volume so that roughness level stays at IRI = 9 m/km, and complete routine maintenance.

RESU

This code represents the periodic maintenance tasks for unpaved roads which includes resurfacing the roadway, reinforcement of the shoulders, and correction of the slopes.

REHABILITATION COSTS PER KILOMETER

On the basis of the tasks to be undertaken in the rehabilitation, the costs per kilometer were determined for each type of rehabilitation; these values served as a basis for the economic evaluation. They are as follows:

RL01	U.S. $20,000/km–$25,000/km
RP01	U.S. $25,000/km–$35,000/km

Both the routine maintenance and the periodic maintenance tasks will be directly calculated by the program on the basis of the unit costs indicated before. Detailed cost estimates will be obtained at the project level using an appropriate rehabilitation design method, such as the one presented by Queiroz et al. (7).

USE OF HDM–III MODEL

HDM–III is the result of a long series of experiments and investigations undertaken in Brazil, Kenya, India, and the Caribbean, which simulates the conditions and the costs of a road's life cycle as a function of its characteristics, traffic volume and weight, environmental conditions, and maintenance operations. Through the benefits resulting from reduced vehicle operating costs, travel time, and other external benefits, the model provides an economic evaluation for each construction or maintenance alternative, expressed in the following economic criteria: net present value (NPV), internal rate of return (IRR), and first-year benefit (FYB). The HDM–III model includes a series of submodels: traffic, maintenance level, vehicle operating cost, exogenous factors, and economic evaluation.

Two types of traffic are considered, normal traffic and generated traffic, which could be a result of a temporary or a permanent traffic generator. The exogenous factors, as they are costs to input caused by special circumstances resulting from the improvement of the road, or lack thereof, makes the calculation more real for each possible situation. For example, consider the case of a road whose condition is so poor that it cannot be used for the transportation of agricultural products. In this case, the exogenous factor is input as an annual loss equivalent to the loss in agricultural production.

ALTERNATIVES CONSIDERED

The model simulates the different alternatives of construction, rehabilitation, or maintenance, always in relationship to a base alternative, identified as ALT0 in the analyses. The alternative ALT1 refers to a light or heavy rehabilitation, in accordance with the condition of the road being considered; ALT2 represents the upgrading of a gravel road into paved road, which could be justified on the basis of a traffic volume of about 150 vpd or on other grounds.

Table 5 presents the input data for each road section considered. The analysis period adopted was from 1990 to 2002. Three discount rates were adopted for NPV calculations: 8, 12, and 16 percent.

DATA PROCESSING AND RESULTS

The model processes the input data that are introduced as a function of the diverse parameters on which the model is based, such as road condition and deterioration laws established in the model for each alternative considered. On the basis of the degree of road deterioration, the corresponding vehicle operating costs are calculated, and with this base, the total benefits of the interventions undertaken in each alternative are calculated in relation to the alternative, ALT0, which consists of no intervention other than some light maintenance. Then, the calculated benefits are processed and their values are presented in their real amounts and by the economic indicators relative to the investment to be made for each road section under consideration.

There is a large number of results obtained, and some may be discarded, given the study's objective. The following list is the totality of the results, presented in reports for each alternative considered or each comparison of alternatives:

Report 1—Summary of Maintenance Performed on the Road (by different components),
Report 2—Annual Maintenance Quantities and Costs,
Report 3—Annual Traffic,
Report 4—Road Condition (for different years and for each alternative),
Report 5—Vehicle Operating Costs (for each year and each alternative),
Report 6—Annual Financial Costs (for each alternative),
Report 7—Economic Costs and in Foreign Exchange (for each alternative),
Report 8—Comparison of Alternatives,
Report 9—Summary of the Comparison of Alternatives by Group,
Report 10—Summary of the Comparison of Alternatives by Discount Rate, and
Report 11—Summary of the Costs and Comparisons by Discount Rate.

Table 6 presents examples of vehicle operating costs for a selected road section, obtained from Report 5.

TABLE 6 TYPICAL VEHICLE OPERATING COSTS IN U.S. DOLLARS

	Vehicle type				
	LIG	UTI	CAML	CAMM	CAMP
AV. SPEED (KM/H)	57.1	51.1	43.4	39.2	32.7
VOC PER VEHICLE-TRIP	15.1	19.3	21.4	35.8	56.5
VOC PER 1000 VEHICLE-KM :					
FUEL	57.5	98.5	47.5	92.5	181.4
LUBRICANTS	5.3	5.3	6.4	7.7	7.7
TIRES	20.3	24.3	78.5	60.3	132.9
SPARE PARTS	135.2	188.0	151.6	293.1	461.4
MAINTENANCE LABOR	3.0	3.4	7.6	11.1	13.3
CREW	.0	5.9	6.9	12.7	15.3
DEPRECIATION	62.2	32.6	99.5	162.8	245.3
INTEREST	18.7	9.8	29.9	48.8	73.6
TOTAL RUNNING COST	302.3	367.7	427.9	689.1	1130.9
OVERHEAD COST	60.5	91.9	107.0	206.7	282.7
TIME VALUE OF PASS	15.8	23.5	.0	.0	.0
TIME VALUE OF CRGO	.0	.0	.0	.0	.0
TOTAL TIME COST	15.8	23.5	.0	.0	.0
REDUCED PASSAB. COST	0	.0	.0	.0	.0
TOTAL COST	378.5	483.1	534.9	895.8	1413.6

Road Section R4-Contuboel-Fronteira (R402)
Year 1990 (Without rehabilitation)

Of all the results obtained for each road section studied, Table 7 presents the capital costs for the implementation of the intervention, the routine maintenance costs, and the total economic benefits.

SENSITIVITY ANALYSIS

For program preparation, the economic indicators for each road section under consideration are presented in Table 8. The economic rates of return are a function of the diverse parameters assumed. So it is advisable to test their sensitivity to variations in the data assumed; consequently, sensitivity tests were done, with new calculations for some road sections, considering levels of traffic 10 percent lower and costs 10 percent higher than anticipated. The results of these tests reveal that these less favorable situations lowered rates of return by only 4 to 8 percent.

BUDGET RESTRICTIONS

The total capital required to rehabilitate all the road sections studied was about U.S. $13.2 million, much above the available financial resources for the road rehabilitation and maintenance program. Therefore, it was necessary to select the road sections to be included in the program and to optimize the interventions to be undertaken with the investment.

The investment to be made on the road network through the present program is budgeted at U.S. $8.7 million for the rehabilitation of the paved and unpaved roads. For maintenance of the network for 5 years (1990 to 1995), there is an available budget of U.S. $7 million that includes purchase of some equipment for the maintenance road brigades (by force account).

OPTIMIZATION OF THE INVESTMENT PROGRAM WITH THE EBM MODEL

The EBM developed by the World Bank (*1*) can be used in conjunction with HDM–III to optimize a road maintenance and rehabilitation program under budget restrictions. The analysis is based on three distinct phases that are described as follows.

Project Data

These are the input data for the different road sections under consideration (investment units), which should include the annual values of economic costs and benefits of a base alternative. The values corresponding to the partial investments should be indicated for each intervention as well as the year it was implemented. These data can be directly obtained through a directory created by HDM–III.

Problem Data To Be Studied

These are the input data for the problem to be studied and which will define the optimization model to be undertaken. These input data essentially establish the periods to be considered in the previously established budget and the restriction levels of the financial resources as well as the discount rate to be considered in the processing of the NPV of each road section under consideration.

Optimization

In this phase, the model examines the available financial resources and, through the NPV resulting from the imple-

TABLE 7 CAPITAL COSTS, ROUTINE MAINTENANCE COSTS, AND TOTAL ECONOMIC BENEFITS IN MILLIONS OF U.S. DOLLARS

ROAD No.	Section	LENGTH km	RUN CALC. REF.	CAPITAL COSTS * ALT1	CAPITAL COSTS * ALT2	RECURRENT COSTS ** ALT1	RECURRENT COSTS ** ALT2	TOTAL ECONOMIC BENEFITS*** ALT1	TOTAL ECONOMIC BENEFITS*** ALT2
PAVED ROADS									
N1	Biss-Jugud	47	N100	1,650	-	0,403	-	22,446	-
N2	Safi-J.Lan	8	N200	0,160	-	0,079	-	1,319	-
N1	Jugu-Bamba	62	N1A0	1,370	-	0,648	-	2,110	-
R9	Bamba-Xime	11	R900	0,220	-	0,104	-	0,210	-
R7	Catio-Impu	18	R700	0,450	-	0,170	-	0,387	-
L35	Jembe-Cadi	11	L350	0,220	-	0,102	-	0,202	-
R3	Manso-Biss	25	R300	0,750	-	0,234	-	1,323	-
UNPAVED ROADS									
N1	Caium-Buru	16	N101	0,320	0,560	0,231	0,081	1,096	1,890
R4	N1-Contubo	19	R401	0,380	0,665	0,293	0,092	4,024	4,454
L24	N1-Galomar	24	L240	0,420	0,840	0,369	0,116	3,948	4,134
R4	Cont-Front	40	R402	0,800	-	0,540	-	3,247	-
R8	N.Sin-S.Jo	13	R800	0,195	-	0,164	-	0,273	-
N2	Mamp-Guile	30	N202	0,450	-	0,368	-	1,103	-
L18	N1-Gambiel	37	L180	0,740	-	0,437	-	2,503	-
L24	Gal-Dulomb	20	L24A	0,400	-	0,236	-	1,112	-
R7	Buba-Catio	52	R7A0	1,040	-	0,662	-	1,946	-
N2	Enxud-Buba	73	N201	1,094	-	0,892	-	4,656	-
R1	Man-Bafata	64	R101	0.960	-	0,798	-	3,482	-
N3	Ingore-Far	66	N301	0,890	-	0,718	-	2,272	-
R1	Binar-Biss	24	R1A0	0,240	-	0,261	-	0,946	-
L30	Ban-Empada	13	L300	0,195	-	0,160	-	0,271	-
L16	Bafa-Gamam	13	L160	0,195	-	0,158	-	0,269	-

* Capital costs during the 1990-1995 period
** Recurrent costs during the analysis period (1990-2002)
*** Total economic benefits for the analysis period

mentation of each road section, selects the group of alternatives that leads to the optimal NPV.

INPUT DATA

In this study, the project input data were directly generated by HDM–III. With reference to the problem data to be studied, two periods were considered for the available resources: the first between 1990 and 1995, which is more important for the program being evaluated, and the second period from 1995 to the end of the calculations in 2002. The adopted discount rate was 12 percent. The financial resources available, as mentioned before, were used as the limiting budget. Additional analyses using several budget levels indicated that the optimal budget level differed by only about 5 percent from the available budget.

ROAD REHABILITATION AND MAINTENANCE PROGRAM

From the input data directly received from the HDM–III model and other data input directly, the EBM model gives as output (a) a summary of the input data; and (b) the rehabilitation alternative selected for each road section, its NPV, the ratio of NPV to cost, and the optimal budget level. The road rehabilitation program thus generated would have a total NPV of U.S. $52.3 million. However, it was deemed necessary to include in the program some predetermined choices to accommodate special conditions, as follows: (a) the R7 road section between Catio and Impungueda (R700), because it provides access to new river ports; (b) the R8 section between Nova Sintra and S. Joao (R800), because of the construction of river ramps to be initiated shortly. This procedure will allow the operation of a ferry boat, thus heightening the importance of this road.

Forcing these interventions into the road rehabilitation program yields a total NPV of U.S. $48.8 million, thus representing a reduction of about 7 percent from the optimal solution given initially by the EBM model. The resulting 5-year road rehabilitation program is presented in Table 9, which presents all the road sections to be rehabilitated.

CONCLUSIONS

Guinea-Bissau presently suffers from an extremely dilapidated infrastructure, and roads are no exception. The gen-

TABLE 8 NPVs, FIRST-YEAR BENEFITS AND INTERNAL RATE OF RETURN

ROAD No.	Section	LENGTH km	RUN CALC. REF.	NET PRESENT VALUE a 12% ALT1	ALT2	FIRST YEAR BENEFIT ALT1	ALT2	INTERNAL RATE OF RETURN ALT1	ALT2
PAVED ROADS				US$x10^6		%	%	%	%
N1	Biss-Jugud	47	N100	9,276	-	74.1	-	134.4	-
N2	Safi-J.Lan	8	N200	0,496	-	58.2	-	61.8	-
N1	Jugu-Bamba	62	N1A0	0,434	-	22.7	-	21.4	-
R9	Bamba-Xime	11	R900	0,013	-	15.2	-	14.0	-
R7	Catio-Impu	18	R700	(0,003)	-	11.6	-	11.8	-
L35	Jembe-Cadi	11	L350	0,011	-	15.1	-	13.9	-
R3	Manso-Biss	25	R300	0,253	-	18.7	-	20.2	-
UNPAVED ROADS									
N1	Caium-Buru	16	N101	0,419	0.660	49.1	34.2	40.3	34.6
R4	N1-Contubo	19	R401	1,854	1,889	77.2	48.7	83.6	55.4
L24	N1-Galomar	24	L240	1,798	1,647	68.3	38.2	75.2	43.6
R4	Cont-Front	40	R402	1,139	-	35.8	-	33.3	-
R8	N.Sin-S.Jo	13	R800	0,051	-	14.6	-	17.2	-
N2	Mamp-Guile	30	N202	0,333	-	24.3	-	27.7	-
I.18	N1-Gambiel	37	L180	0,874	-	37,6	-	41.2	-
L24	Gal-Dulomb	20	L24A	0,364	-	32.6	-	35.2	-
R7	Buba-Catio	52	R7A0	0,428	-	7.6	-	20.9	-
N2	Enxud-Buba	73	N201	1,648	-	54.3	-	48.1	-
R1	Man-Bafata	64	R101	1.156	-	44.7	-	39.2	-
N3	Ingore-Far	66	N301	0,628	-	31.1	-	27.8	-
R1	Binar-Biss	24	R1A0	0,332	-	46.6	-	52.0	-
L30	Ban-Empada	13	L300	0,050	-	14.5	-	17.1	-
L16	Bafa-Gamam	13	L160	0,048	-	14.4	-	16.9	-

TABLE 9 ROAD SECTIONS SELECTED FOR THE REHABILITATION AND MAINTENANCE PROGRAM

ROAD TYPE		ROAD SECTION	KM	CODE
	N1	Bissau (Rot.Aerp.)-Jugudul	47	N100
	N2	Safim-Joao Landim	8	N200
PAVED	N1	Jugudul-Bambadinca	62	N1A0
ROADS	R9	Bambadinca-Xime	11	R900
	R7	Catio-Impungueda	18	R700
	L35	Jemberem-Cadique	11	L350
	R3	Mansoa-Bissora	25	R300
	N1	Rio Caium-Buruntuma	16	N101
	R4	N1-Contuboel	19	R401
UNPAVED	L24	N1-Galomaro	24	L240
ROADS	R4	Contuboel-Fronteira	40	R402
	L24	Galomaro-Dulombi	20	L24A
	R8	Nova Sintra-Sao Joao	13	R800

erally poor condition of paved and unpaved roads is seriously affecting transport costs between agricultural production areas and river ports or consumption centers. A 5-year road rehabilitation program to produce an acceptable length of roads in good condition has been discussed.

The program economic analysis was carried out with HDM–III. Rates of return and NPV values of alternative improvement strategies both of paved and unpaved road sections were given. The optimum set of road sections and corresponding maintenance options were then obtained with application of the EBM. The improved road network will contribute to raise the country's level of economic activity, by means of reduced freight and passenger transportation costs, more efficient transportation of goods to consumption centers and ports, increased mobility for the population as a whole, easier access to medical and social services, and improved road safety for vehicles and pedestrians.

The EBM, with the capability of using as input the economic data generated by the HDM and its flexibility in terms of budget levels and budgeting periods, is an excellent instru-

ment for long-term planning of interventions on the road network.

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Towed Graders and Tractor-Based Maintenance of Low-Volume Roads

R. C. Petts and T. E. Jones

The historical development of equipment for constructing and maintaining the surface of earth and gravel roads is reviewed. The use of tractor-drawn graders and drags is emphasized. The case for tractor-based road maintenance operations for these roads in developing countries, instead of the normal motor grader and truck-based operations, is also developed. For many secondary and tertiary road networks, routine and some periodic maintenance could be carried out using wheeled agricultural tractors as the sole power units towing mechanical graders, gravel haulage trailers, water bowsers, and rollers, as required. The operations could be supported by local labor for such activities as gravel excavation, loading and unloading, spreading, and drainage maintenance. The road maintenance could be carried out by direct labor (force account) organizations or contracted out to the private sector. Contract agricultural work is already well established in many developing countries and road maintenance and improvement activities could extend the type of work carried out by tractor-owning contractors. Tractor-based road maintenance operations would be significantly cheaper in terms of capital and running costs. The logistic and support problems of such operations would also be less than for current motor grader-based operations. Tractor-based road maintenance operations would make more effective use of the limited financial and other resources available to road maintenance organizations in developing countries.

In many developing countries, the motorized grader is the principal equipment item for maintaining unpaved roads. However, the availability of this equipment is low because of inadequate funding resources, mainly for spare parts and maintenance facilities, and the shortage of the trained manpower required to maintain the equipment. (The terms availability and utilization frequently used in the text refer to equipment that is available for maintenance works and is utilized in maintenance activities. Utilization is expressed as a percentage of the time that the equipment is available for work.) This low level of availability has led, particularly in African countries, to inadequate maintenance operations on the majority of the unpaved road networks (*1*).

Recent research (*2*) suggests that for many secondary and tertiary road networks routine and some periodic maintenance could be carried out using wheeled agricultural tractors as the sole power units, towing mechanical graders, gravel haulage trailers, water bowsers, and rollers as required. The operations could be supported by local labor for such activities as gravel excavation, loading and unloading, spreading, and drainage maintenance. The maintenance could be carried out by direct labor (force account) organizations or contracted out to the private sector. Contract agricultural work is already well established in many developing countries and road maintenance and improvement activities could provide an extension to the type of work carried out by tractor-owning contractors.

The historical development of equipment for constructing and maintaining the surface of earth and gravel roads is reviewed. The use of tractor-drawn towed graders is emphasized and towed drags are briefly described. The case for tractor-based road maintenance operations for these roads in developing countries, instead of the normal motor grader and truck-based operations, is also described.

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HISTORICAL DEVELOPMENT

Tractor or vehicle-drawn graders have been used for grading the surfaces of unpaved roads at least since 1909 (see Figure 1). Initially, they had solid metal wheels and were normally drawn by tractors (see Figure 2) although some models were self-propelled, as shown in Figure 1. Very early graders often incorporated complicated mechanical devices for carrying out activities in addition to surface grading. These included the construction of miter drains (turn outs), side drains, and side slopes, as shown in Figures 3 and 4. Early road construction techniques also included graders towed in tandem and the use of tractor-drawn trailers with side-tipping facilities for the transportation of gravel wearing course or fill material, as shown in Figures 5 and 6. This type of road construction equipment was used quite extensively in most countries until the mid-1930s when the motorized grader was developed from a hybrid tractor fitted with a full-width moldboard (Figure 7).

In developing countries, particularly in the early 1950s, the introduction of the motorized grader, often funded by developed countries or donor agencies, led to a decline in the use of tractor-towed grading equipment. However, subsequently a range of factors meant that the operation of motorized graders in developing countries became problematic, resulting in extremely low availability of equipment. In many cases, the end result is that road maintenance is simply not carried out, particularly on important and highly trafficked routes.

The low availability of maintenance equipment is one of the principal reasons for the poor condition of a substantial amount of the unpaved road networks in Africa (*1*). More recently, this situation has led to countries looking at cheaper and more reliable alternatives. In the last 20 years, a number of countries, particularly in Africa, have started to utilize tractor-towed graders for surface treatment of gravel roads (Figure 8) and, more recently, tractor-towed compactors, water

FIGURE 1 Self-propelled grader, 1919.

FIGURE 2 Road leveler, 1919.

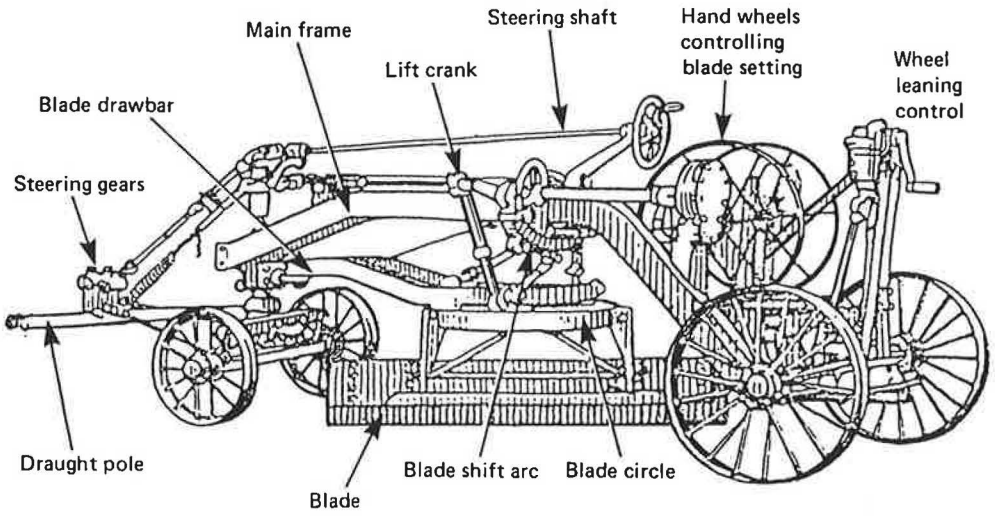
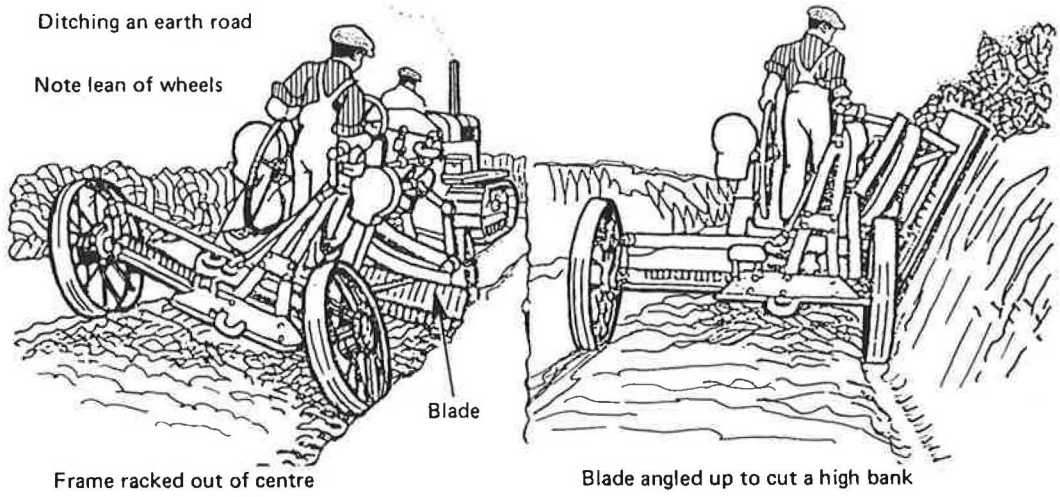


FIGURE 3 Basic towed grader, 1920.



This type of Grader has a blade that can be set to any angle on either side to form road surfaces, ditches or banks. This one is tractor-hauled, but some are self-propelling

FIGURE 4 Ditching and side slope techniques.

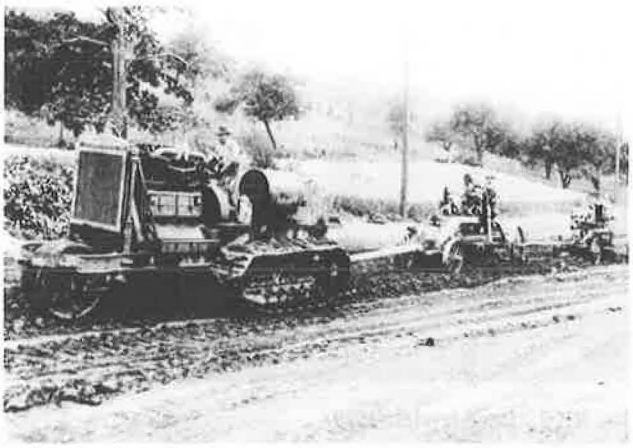


FIGURE 5 Tandem towed grader, 1920.

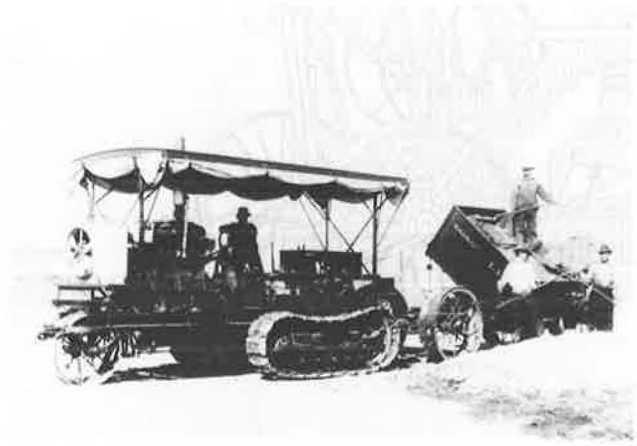


FIGURE 6 Early side-tipping trailers.

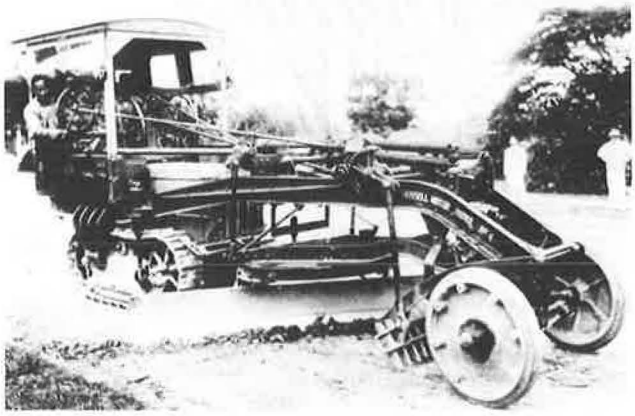


FIGURE 7 Prototype motorized grader, 1926.

FIGURE 8 Grader in use in Africa in 1960s.

bowsers, and trailers. Some countries are now exploring the feasibility of operating maintenance units based entirely on tractor-towed systems. One example is Thailand, where the Office of Accelerated Rural Development (ARD), which is responsible for the construction and maintenance of 19 000 km of unpaved road, has already started to implement such a system (Figures 9 and 10).

The development of mechanical drags for leveling unpaved roads has changed only slightly from the 1920s when the prime mover was a horse as shown in Figure 11. Early models of the drags shown in Figures 12–14 are basically the same as those used today in developing countries. They are effective in reducing surface irregularities on gravel roads and on low-trafficked routes and are often the most appropriate solution. Current usage and models are discussed in detail elsewhere (*2*).

CURRENT PROBLEMS OF MOTOR GRADER–BASED ROAD MAINTENANCE IN DEVELOPING COUNTRIES

In many developing countries, the principal item of routine maintenance equipment for earth and gravel roads is the motor grader. It is also sometimes used to reshape the unsealed shoulders of paved roads.

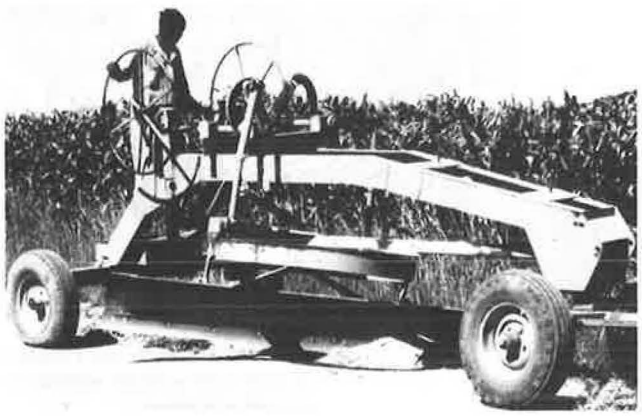


FIGURE 9 Towed grader currently utilized in Thailand.

FIGURE 10 Tractor-towed compactor.

The motor grader is used for reshaping the running surface, removing corrugations, and cleaning the side and miter drains (turn-outs). The machines typically work alone, operating from a road camp base. Grader operations are usually supplemented by labor gangs also working from road camps.

Although the support operations are manual, a truck is usually required to transport the gang from the road camp to the worksite each day. Other specialist equipment such as bulldozers, front-end loaders, and self-propelled rollers and bowsers are used for the provision of gravel surfacing, which is required for routine patching and periodic regraveling. Haulage is normally carried out using tipping trucks.

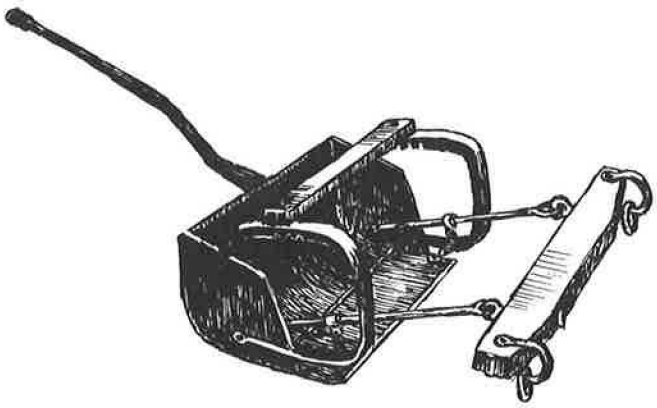


FIGURE 11 Horse-drawn scraper.

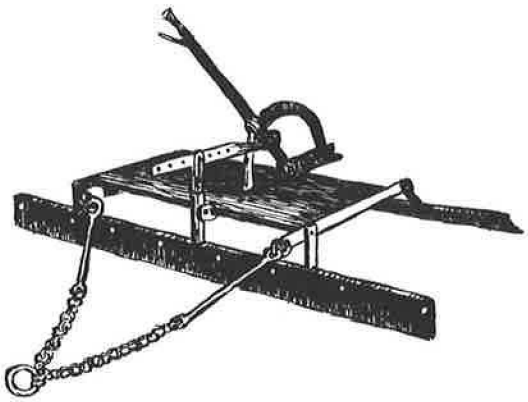


FIGURE 12 Two-bladed adjustable road drag.

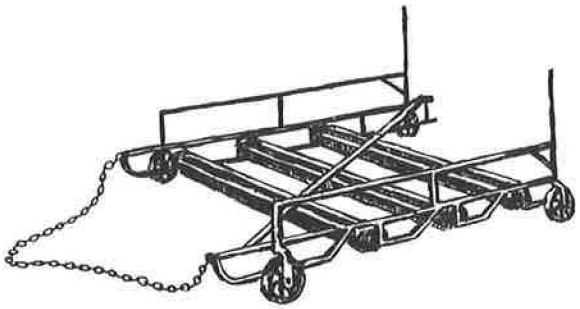


FIGURE 13 Road hone.

These equipment-based operations were set up before the global oil crises of the 1970s. Before these events, equipment was relatively cheap and road authorities in developing countries did not face the serious problems of procurement and equipment support that many of them face today.

The operating environment of road authorities has changed considerably over the last 20 years. Motor graders, front-end loaders, and bulldozers are now expensive and specialist items of equipment, ideally suited for use by medium or large contractors in developed countries. These organizations are able to closely support their equipment on construction sites with good site workshops and mobile workshop facilities. Highly skilled mechanics and operators, readily available spares, and technical support enable the full potential of the machines to be realized in these circumstances.

These conditions rarely exist in developing countries. Typical problems currently encountered by the road authorities include the following:

- Equipment working individually at long distances from workshop facilities.
- Poor workshop facilities—mobile workshops rarely exist.
- Lack of necessary trained workshop personnel able to manage or carry out the specialist servicing and repair work.
- Cumbersome procurement procedures for spare parts and replacement equipment.

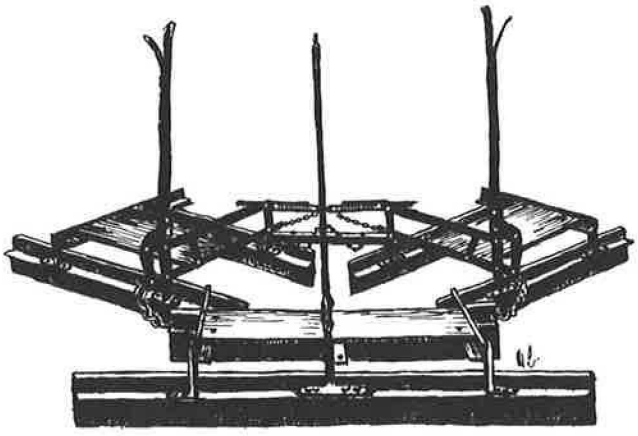


FIGURE 14 Three-way road drag.

- Wide variations in models and makes of equipment because of restrictive tendering (usually lowest initial cost) procedures, exacerbating spares procurement and stocking problems.
- Foreign exchange shortages exist in many developing countries so that road authorities and equipment agents have difficulties in procuring equipment and spares from abroad. These organizations rarely get priority allocations of available foreign exchange, particularly for road maintenance equipment.
- The national fleets of the individual equipment models are relatively small so that it is uneconomic or impractical to maintain extensive spares stocks in a country, either with the road authority or the local equipment agents.
- A lack of appreciation of the true cost of operation of sophisticated imported equipment because of the funds accounting, rather than total cost accounting, systems used.
- Absence of effective management information systems.
- Lack of accountability or incentives to perform in many road authorities.

With these constraints, it is not surprising that the road authorities cannot even approach the equipment availability rates achieved by developed country contractors. Instead of achieving rates of 60 to 70 percent, or more, availability (where reliable records exist) is typically of the order of 20 percent for road maintenance equipment (*3*). The cost of owning and operating equipment at these low rates is uneconomic, as Table 1 indicates.

Interventions have been made with the assistance of various external agencies over recent years to try to counter the above-mentioned constraints. Provision of new equipment, spares, workshops, or training have often brought short-term improvements in equipment performance. However, the improvements have usually not been sustained, because of inability to provide a comprehensive solution to the range of influential factors.

It is necessary to radically rethink the approach to maintenance of unsurfaced roads in developing countries. Less reliance must be placed on the use of sophisticated, expensive, imported equipment. A new approach must be made that makes better use of the locally available technologies and resources.

ADVANTAGES OF TRACTOR-BASED OPERATIONS

Wheeled agricultural tractors are mechanically simple compared to conventional road maintenance equipment. However, units are now commonly available with power of up to 100 hp (75 kW) and above, with either two- or four-wheel drive. Tractors of 50 to 80 hp are adequate for hauling towed graders of about 2 tonnes for light grading. Heavier (4-tonne) towed graders usually require tractors of 80 to 100 hp and four-wheel drive configuration. These can be used for heavy grading.

Agricultural tractors are also widely available in developing countries. Table 2 presents details of annual sales of agricultural tractors in Kenya compared to the total fleet of motor graders in the country.

The capital cost of a four-wheel-drive, 100-hp tractor is typically about one-quarter that of a 125-hp motor grader. Even including the cost of the towed grader unit, the tractor combination is much less than half the price of the maintenance motor grader. Investment (interest on capital) costs are correspondingly lower.

The support and running costs for agricultural tractors and towed equipment are also significantly lower than for conventional road maintenance equipment. Furthermore, the foreign exchange requirement for both capital and running costs would be reduced substantially below current levels.

The availability of any item of plant in developing countries is affected considerably by the availability of spare parts. Major problems that occur with motor graders usually involve the hydraulics, clutch, or transmission systems, which are complex and difficult to correct. The problems are often exacerbated, as few of the appropriate spares are stocked because

TABLE 1 EFFECT OF UTILIZATION ON THE OPERATING COSTS OF AN ITEM OF EQUIPMENT

Typical hourly costs for equipment (US$)			
Actual annual utilisation (hours) % of working year	1000 57	500 29	250 14
Fixed costs Interest, depreciation workshop and salaries	20	40	80
Variable costs Fuel, lubricants tyres, maintenance repairs cutting edges	15	15	15
Total hourly cost	35	55	95

Note: Working year calculated as 1750 hours (250 working days @ 7 hours)

TABLE 2 AVAILABILITY OF AGRICULTURAL TRACTORS AND MOTOR GRADERS

Agricultural Tractors: National Sales (All Manufacturers)					
	40-50hp	50-80hp	80-110hp	>110hp	ALL
1986	164	487	327	94	1072
1987	153	516	295	23	987
1988	325	404	318	46	1093
1989	242	529	351	66	1188
	884	1936	1291	229	4340

Motor Graders: Estimated Total National Fleet: All Manufacturers	
	No
Ministry of Public Works	350
Other Organisations	150
Estimated national fleet	500

Source: Motor Trade Association

of their high cost. Even on projects utilizing new equipment, there are often insufficient or inappropriate spares supplied. The importation of spare parts usually involves foreign exchange allocation and this frequently introduces delays in procurement. Repairs to tractors, however, are usually more straightforward, being mostly associated with difficult starting or contaminated fuel. In the case of the tractor-towed grader units, not only are spare parts more interchangeable with motor tractors but, because of the size of the agricultural industry in most developing countries, parts are more commonly available. Towed grader performance, particularly in Zimbabwe where over 200 are in use, and also in Zambia and Malawi, indicates that their availability is over 90 percent and spare parts do not present a problem. Tractors and towed graders also require a lower level of expertise than motor graders for minor repairs and overhauls. Tractor-based equipment can normally be expected to provide availability of 2 to 3 times that of the specialist road maintenance equipment.

Adoption of agricultural tractors as the basic power source for maintenance grading operations also allows flexibility to carry out other maintenance tasks. In addition, the tractors can be used to tow gravel haulage trailers for regraveling. The Kenya Rural Access Roads and Minor Roads Programs have used tractor-trailer combinations to haul gravel for the construction and periodic maintenance regraveling of a road network of over 9 000 km. Operations are currently planned for expansion to a network of 12 500 km (*4*).

Tractors can also tow rollers and water bowsers to achieve better compaction of gravel surfacing. Even workshop and labor accommodation could be towed in tandem with equipment between remote worksites to minimize mechanical support problems and avoid the need to return to base camp each evening. It would be possible to set up mobile maintenance units with 2 or 3 tractors based on the concept shown in Figure 15.

The tractor-based operations could be supported by local labor for such activities as drainage maintenance, gravel excavation, loading, unloading, and spreading, where appropriate.

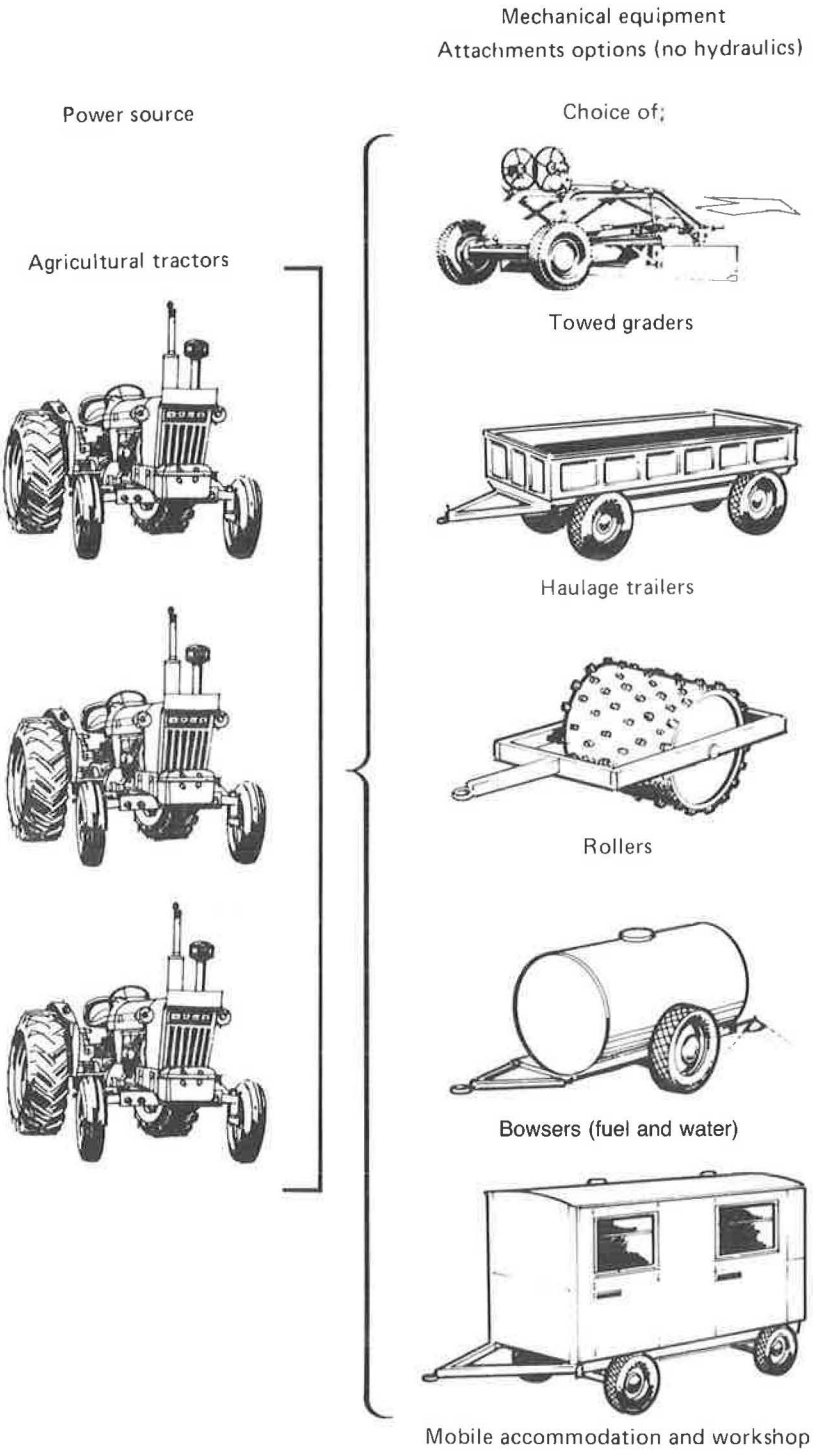
An important consideration is that all of the attachments—towed graders, gravel haulage trailers, rollers, bowsers, workshop, and mobile accommodation—could be built and therefore easily maintained in developing countries.

The low capital and running costs and ease of support of tractor-based equipment means that local contractors can be attracted to road maintenance activities using this type of equipment.

Road maintenance contracts could complement contract agricultural activities already established in many developing countries, particularly where the latter have a limited season.

AVAILABILITY OF MAINTENANCE EQUIPMENT

The level of maintenance achieved on any road is mainly a function of the allocation or availability of staff, fuel, and equipment. Of these three factors, it is often the availability of the equipment that limits the frequency of maintenance operations. For unpaved roads in developing countries, the



Notes. Units can be towed in tandem between worksites
Number of each item according to maintenance requirements

FIGURE 15 Flexible technology.

motor grader is the most commonly used item of maintenance equipment. However, because of the factors already discussed they tend to have a low level of availability, particularly when compared to other types of plant used in both construction and maintenance activities (*5–7*).

Kenya

The then Kenyan Ministry of Transport and Communications (MOTC) estimated in 1982 and 1983 that their motor graders had a typical availability rate of 60 percent. However, in the Kenyan regraveling program, this figure was found to be over-optimistic. In 1981, availability rates on the regraveling projects in Western and Nyanza provinces were found to average 39 percent for 13 motor graders, 8 of which were less than 2 years old. In the following year, the average had dropped to 31 percent, as indicated in Table 3.

Equipment records were not available for the other geographical areas utilized in the study, but availability of motor graders elsewhere is unlikely to have been better than these rates. All of the regraveling projects had the expertise of mechanical superintendents who were able to provide a high standard of plant maintenance. The main complaint of the superintendents was the inexperience of the plant operators and the lack of suitable spare parts. Many of the motor grader spares stocked at the various regraveling depots were items seldom required for field operations.

At the same time, there were over 100 motor tractors in use with the MOTC. These varied in size from 40 to 80 hp. Records are again sparse, but estimates of availability from regional mechanical depots ranged from 60 to 80 percent. In addition, some 90 tractors, nominally of 48 hp, were utilized by the Rural Access Roads Program (RARP) for gravel haulage. Since December 1981, the RARP kept detailed records of plant availability, some of which are indicated in Table 4, which presents availability and utilization rates for a 6- to 8-month period.

Elsewhere in Africa

The TRRL found in Ghana that motor graders were only available for 10 percent of the time, compared with 30 percent for agricultural tractors utilized by the same organization (*5*). These figures are particularly low, even allowing for the conditions generally met in developing countries. More typical levels of availability found elsewhere are 50 and 60 percent,

TABLE 3 MOTOR GRADER AVAILABILITY IN KENYAN REGRAVELING PROJECTS (PERCENT)

MANUFACTURER	TOTAL No	1981	1982	Average
A	8	38	25	32
B	3	33	33	33
C	2	50	50	50
Weighted average		39	31	

TABLE 4 TRACTOR AVAILABILITY IN THE RURAL ACCESS ROAD PROGRAM, 1981 TO 1982

Unit	Working Days	Days Available	Days Worked	Availability (per cent)	Utilisation (per cent)
Kwale 1	120	55	50	46	42
Kwale 2	138	78	38	60	28
Embu 1	108	84	49	78	45
Embu 2	108	72	51	67	47
Uasin Gishu 1	156	90	32	58	21
Uasin Gishu 2	156	97	75	62	48
W. Pokot 1	156	130	41	83	26
W. Pokot 2	156	130	34	83	22
AVERAGE	137	92	46	67	35

Even if the exceptionally high rates at West Pokot are excluded, the average availability is still over 65 per cent.

respectively (*6*). In general, the higher figures are obtained by contractors, whereas government agencies normally have lower levels of availability, some substantially lower (*7*).

In 1982, the International Labor Office (ILO) produced a list of availability and utilization rates in 10 African countries (*6*) for different items of equipment used on road maintenance operations. The rates quoted for availability of motor graders and large agricultural tractors were approximately 50 and 60 percent, respectively. The reason given for the relatively high level of grader availability was that, in a number of the countries, the equipment was new. However, the report suggested that, if a thorough analysis was carried out over a longer period, the figure would be lower.

Generally, records of equipment availability in developing countries are poor and rates are often based on theoretical estimates rather than on equipment records. For example, the World Bank reports annual availability of 1,824 hr for motor graders (*1*). This amount is equivalent to over 7 hr per day for 250 working days a year, which is essentially 100 percent availability, which is highly unlikely. In personal communications, service managers of motor grader suppliers in Kenya, Zimbabwe, and Botswana suggested that the figures quoted in the ILO report for motor graders were higher than what would normally be met in practice in developing countries, with the exception of new equipment or equipment working under ideal conditions.

CURRENT USAGE OF TOWED GRADERS

Zimbabwe

Local manufacturers in Zimbabwe currently fabricate a 2-tonne towed grader, with a total production of over 500 to date since 1960. Of these, some 200 are currently utilized in Zimbabwe by the three authorities responsible for gravel roads. These are the Ministry of Transport, the Association of Rural Councils, and the District Development Fund. A number of other African countries—namely Malawi, Zambia, and, to a lesser extent, Ghana, Tanzania, and Kenya—also use the Zimbabwe-manufactured grader, which is normally towed with a 50-hp agricultural tractor. These graders are generally used for maintaining gravel roads with traffic volumes of up to 200 vehicles per day but are also used in Zimbabwe for light construction activities. These towed graders have proved to be extremely reliable with a minimal requirement for spare parts. The experience of Zimbabwean authorities and those of other countries in southern and central Africa means that this type of grader will be in continuous use for some time.

Thailand

The Office of Accelerated Rural Development in Thailand, which is responsible for the construction and maintenance of 19 000 km of rural roads, is currently using the Zimbabwe towed graders on an experimental basis. If their performance is found to be acceptable, then almost certainly the towed grader will be manufactured locally. There were some minor problems initially in Thailand related to the physical effort required to operate the graders but these have been overcome with the use of more efficient gearing for the controls. The experimental study in Thailand indicates that the performance of the towed grader in reducing surface roughness is similar to that achieved using the motorized graders.

Kenya

Although the 2-tonne Zimbabwean grader provides a satisfactory performance, some engineers feel that an increase in weight would make it more adaptable for construction purposes and for maintaining heavy clay soils or coarse gravels, e.g., volcanic and cinder materials. In Kenya, a local manufacturer is currently producing a heavier towed grader of approximately 4 tonnes. These are fully trailed units with greater load transfer on the tractor driving wheels, while the longer distance between the hitch point and rear wheels ensures a more even grade (Figures 16 and 17).

Elsewhere

In Great Britain, towed graders are available but they are normally only utilized for light grading, e.g., on sports grounds. These are the Hallam and Simba graders. They have been

FIGURE 16 Prototype heavy grader on trial near Nairobi.

FIGURE 17 Prototype heavy grader on trial on clay soils.

used overseas, but the main drawback is that their design is more appropriate for developed country circumstances that could result in problems similar to those associated with the motor graders in developing countries, i.e., foreign exchange constraints on imported spare parts.

Feedback from many engineers responsible for the maintenance of rural roads in developing countries highlights the real need for alternatives to the motorized grader for reducing surface roughness and restoring camber. The development of the Zimbabwean or Kenyan towed grader should provide the solution.

COSTS OF GRAVEL ROAD MAINTENANCE

The operating cost of grading equipment will have a significant effect on the cost of gravel road maintenance. Table 5 presents typical operating costs per hour for a motor grader and tractor-towed grader combination. All costs are based on January 1987 figures and although they are Kenyan in origin they will be typical of costs in many African countries. The main differences that may occur are the local costs of labor, fuel, and tires.

The condition of the gravel road surface dictates how many passes of the grader are required for each maintenance operation. Some countries stipulate a minimum of five passes in their operation manuals. In practice, the poor initial condition of the road makes more passes necessary. This situation is because of the low availability of motor graders and other factors making maintenance infrequent. Assuming five passes, it then follows that, at an average speed of 2 km/hr, it will require 5 hr of continual operation to complete 2 km. Normal daily working periods are 8 hr, but the available time for grading each day is unlikely to be greater than 5 hr, taking into consideration traveling to site, refuelling, and downtime. Therefore, the output of the motor grader is estimated at 2 km per day. This result is confirmed by the Kenyan MOTC, who, in 1984, calculated that their motor graders' output was of this order. Therefore, actual daily costs to grade 2 km of road are as follows: 8 hr at U.S. $45.55 = U.S. $364, i.e., U.S. $182/km.

One of the results of the TRRL study in Kenya showed that a tractor-towed light grader unit could maintain approximately 1 km of road per day. The unit requires 8 hr of operating time, so the costs of maintaining 1 km of road with these units will be as follows: 8 hr at U.S. $12.18 = U.S. $97/km.

However, the availability of a motor grader in Kenya was only 30 percent, whereas motor tractor availability is about 60 percent. Therefore, despite the higher potential output of the motor grader, the same length of road network could be maintained in a given period by the same number of units of either equipment, but at a reduced cost of U.S. $97/km if the tractor-towed grader units are used. However, if the availability of motor graders is reduced, the unit costs of operation will increase because of the need to reapportion depreciation and interest charges, as Table 1 shows. Because of the larger capital costs of motor graders compared with tractor-towed grader units, the increase in hourly costs will also be larger, making tractor-towed grader units even more competitive. Thus, for maintaining a properly constructed road, the tractor-towed grader unit can be considered as an alter-

TABLE 5 HOURLY OPERATING COSTS OF MAINTENANCE EQUIPMENT, 1987

	Motor Grader	Tractor/towed grader	Tractor/drag unit
Item	Costs/hr (US$)	Costs/hr (US$)	Costs/hr (US$)
Depreciation	16.45	2.45	1.87
Interest	9.05	0.98	0.66
Fixed costs	25.49	3.42	2.54
Fuel	7.23	2.26	2.26
Lubrication	0.55	0.22	0.18
Tyres	2.11	0.44	0.35
Maintenance	5.72	2.10	1.98
Cutting edges	0.95	0.29	0.46
Operators/ salaries	3.51	3.51	1.76
Running costs	20.06	8.76	6.98
Total costs	45.55	12.18	9.52

native to the motor grader. The overall output per unit is likely to be the same and the cost of operation is significantly less. However, light towed grader units cannot be used to maintain badly deteriorated roads for which a motor grader or heavy towed grader will still be needed.

This is not the case for the tractor-mechanical drag units that, although capable of reducing surface roughness, pothole depths, and corrugations, will not restore the camber of the road. The standard of work achieved will also be lower. Therefore, tractor-drawn drags have not been directly compared to the other types of maintenance equipment.

In Kenya, there are currently some 25 000 km of engineered gravel roads. If maintenance strategies utilizing tractor-towed grader units were adopted instead of motor graders, then for a grading frequency of twice a year and a cost saving of U.S. $85/km, over U.S. $4.3 million could be saved annually. It is also important to realize that, by using less complex equipment that is less reliant on imported spare parts and foreign exchange constraints, the desired maintenance is far more likely to be achieved.

CONCLUSIONS

Maintenance of earth and gravel roads in developing countries is normally based on the use of motor graders. Currently, there are major problems with the operation and maintenance of this relatively sophisticated equipment.

In recent years, the power range of wheeled agricultural tractors has increased and these units are now widely available in developing countries.

There is potential for the development of tractor-based road maintenance at a lower cost and more effective alternative to motor graders. Foreign exchange requirements for new equipment and spares could be significantly reduced.

Tractor-based operations would offer flexibility with the same simple power source used to tow a variety of heavy-duty work tools. The equipment could be supported by local labor for certain routine and periodic maintenance activities. Such an approach would encourage the greater use of local resources and development of local manufacturing capability. The technology would also be suitable for local contractors.

Tractor-based operations now offer a lower-cost, sustainable, and more effective alternative for the maintenance of many roads in developing countries.

ACKNOWLEDGMENTS

Many of the ideas presented here have been developed as a result of collaboration between the TRRL Overseas Unit and other organizations. The TRRL contribution forms part of the research program of the Overseas Unit (head: J. S. Yerrell). The paper is published by permission of the Director of the British Transport and Road Research Laboratory and reproduced by permission of Her Brittanic Majesty's Stationery Office.

Due acknowledgments are made to the Caterpillar Co. of Illinois for some of the early photographs and to David Griffith, Turbomech Ltd. of Nairobi, who is responsible for the development of the Kenyan heavy grader.

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Views expressed herein are not necessarily those of the British government.

Associations of Distress and Diagnosis of Bitumen-Surfaced Road Pavements

José Carlos de O. S. Horta

As a contribution to the implementation of a high-quality and cost-effective road maintenance management system, a catalog of pavement distress and defects was prepared for the low-volume bitumen-surfaced rural roads of Madagascar. The catalog comprises the description, definition, and methods of measuring and rating the different types of distress and defects found in the rural road network as well as checklists of possible causes and guidelines for remedial works. It also comprises a discussion of associations of distress types and their interpretation as clues for proper diagnosis of the actual causes of distress. Not all types of individual distress inventoried in the catalog and discussed in the literature are listed or described. The purpose is to illustrate associations of distress types currently found in low-volume road pavements of Madagascar and other tropical countries and show how identification of certain combinations of distress types can be used for proper pavement maintenance and management. The following associations of distress types can provide clues to diagnosis of distress causes and remedies: raveling and cracking; wide ruts and alligator cracking; longitudinal cracks and settlements along the lower shoulders; crack-bounded settlements; and the association of longitudinal cracks, differential settlements, and undulations. Other associations of distress discussed are bird bath and alligator cracking, eroded-shoulder and longitudinal edge cracking, narrow rut and longitudinal heaving, block cracking of cement bound bases, crescent-shaped cracks and peeling, longitudinal cracks and settlements, and local shearing, as well as corrugations and undulations. Cracked blisters are also discussed.

Engineers concerned with maintenance and rehabilitation of bitumen-surfaced roads should be able to interpret the patterns of distress of individual road sections and to make a diagnosis of the possible causes of actual distress in order to select and implement the most effective remedial works.

Pavement distress types have previously been described and agencies of different countries have published indexes or catalogues of pavement distress and defects (*1–4*).

As a contribution to the implementation of a high-quality and cost-effective road maintenance management system in Madagascar, a catalog of pavement distress and defects was prepared for the low-volume rural road network of that country (*5*). It comprises the description, definition, and methods of measuring and rating 43 different types of distress found on the bitumen-surfaced pavements of the island, as well as checklists of the possible causes and guidelines for remedial works.

The standardization of distress definition and rating is a necessary step for the evaluation and implementation of the proper remedial works. But a catalog or index of distress types alone does not appear to be sufficient for that purpose even when illustrated by photographs of typical examples.

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The essential step for implementation of the proper remedial works is distress diagnosis. For this purpose, destructive testing, comprising verification of pavement thicknesses as well as sampling and testing of pavement materials, is widely used. However, when associations of different types of distress give clues to diagnosis, destructive testing may be minimized and efficiency improved.

The Malagasy catalog also contains a discussion on the significance of different distress associations as a base for well-founded judgment by road engineers and appropriate diagnosis of the causes of distress of road pavements.

In Madagascar, bitumen-surfaced pavements generally comprise a subbase with varying thickness (usually 15 to 30 cm) of selected soil, a 15- to 20-cm-thick processed base (penetration of water-bound macadam or graded crushed stone or soil-cement), and a 2.5- to 5-cm-thick surface layer (hot-mix or surface dressing). Shoulders are usually selected soil with turfing.

PAVEMENT DISTRESS TYPES AND CLASSIFICATION

The different types of distress of bitumen-surfaced road pavements may be classified into three groups: cracking, distortion, and loss of materials.

Cracks resulting from breaking of pavement layers are seldom to be found isolated. Cracks are usually branching out or building families. They may also form particular patterns such as alligator and block cracking.

Distortion of the pavement surface under traffic loading may take the form of bird baths in its initial stages, but usually develops into rutting. It is important to make a distinction between narrow and wide ruts. Narrow ruts have small radii and result from strain in surface or shallow pavement layers, whereas wide ruts with large radii result from strain in the subgrade or deep pavement layers.

Settlements are particular distortions in the form of depressions open to one or both pavement edges. They may result from traffic loading as well as from water action and landslides.

Other types of distortion include corrugations with decimetric wavelengths, longitudinal undulations with metric to decametric wavelengths, collapsed voids, as well as different types of heaving and shearing.

The most popular type of loss of materials is potholing. Potholing is the end result of development of most types of pavement distress. After raveling along their edges, cracks develop into potholes. Raised areas of distortion result in

damage and potholes through dynamic traffic loading. Individual loss of materials, such as stripping, develops into raveling and potholing. Loss of entrapped water and fines or dissolved matter also cause potholes. The causes of potholes are thus multiple and varied, and this type of distress cannot provide clues for diagnosis by itself. In combination with other types, it indicates advanced stages of distress.

Other types of loss of materials are stripping or loss of individual aggregates, raveling and disintegration with loss of binder and aggregate lumps, edge breaking and raveling, peeling, glazing or polished aggregates, bleeding, and loss of water with fines and dissolved matters.

A given type of distress may have multiple causes. Associations of two or three distress types may characterize different processes of road pavement failure, and identification of these associations can be used as a method of diagnosis of causes of distress and failure.

CHARACTERISTIC ASSOCIATIONS OF DISTRESS TYPES

Fifteen characteristic associations of distress types are discussed and illustrated in this section.

Table 1 presents information and indicates possible remedial works for the different associations of distress.

Most distress associations have a unique relationship to diagnosis but some may require additional information for proper diagnosis.

Raveling and Cracking

The association of alligator cracking, disintegration, and loss of aggregates and binder from an undistorted pavement surface is typical of bitumen aging and announces the end of the useful life of the bituminous wearing course (Figure 1). If resealing is not provided in due time, the pavement distress will quickly develop into functional failure by potholing and may require costly rehabilitation works such as recycling, overlaying, or reconstruction.

Bird Bath and Alligator Cracking

The association of alligator cracking with local surface depressions (bird bath) points to localized low bearing capacity of granular pavement structures (Figure 2).

This pattern of distress may form along the wheel paths and develop into wide ruts and alligator cracking.

Wide Ruts and Alligator Cracking

The association of wide ruts and alligator cracking (Figure 3) is the typical distress pattern of fatigue of granular pavements.

Low-volume roads may usually exhibit this type of structural distress where the traffic includes heavy loads. Pavements exhibiting wide ruts and alligator cracking should be reconstructed or strengthened by overlaying or by recycling with particular attention to drainage. In the Malagasy rural roads network, wide ruts and alligator cracking were only present on two roads carrying trucks loaded with chrome ore.

Sometimes wide ruts result from consolidation of open graded or soft pavement materials such as telford and macadam of lateritic boulders (Figure 4). In this case, a leveling course will restore driving comfort.

Narrow Ruts and Heaving

Narrow ruts indicate strain in the upper pavement layers, whereas adjacent heaving points to shearing. Hence, for pavements with adequate thickness this pattern of distress is symptomatic of low-grade materials in the base or wearing course. If the wearing course is thin, as in Madagascar, where hot-mixes usually have thicknesses of 1 in. or slightly more, the cause lies in the base course. Laboratory testing of the base material will provide diagnostic information such as inadequate grading (lack of gravel fraction or excess of sand fraction), high plasticity of fines, or in-service weathering of basaltic natural gravels (Figure 5).

Underdesigned, thin pavements may fail with development of narrow ruts and adjacent longitudinal heaving.

Crack-Bounded Settlements

Crack-bounded settlements are also associated with pavement distress associations.

Branching, multiple parallel cracks or single cracks are actually the intersection of a slide surface with the pavement surface. In some cases, the cracks cross the pavement at variable angles and the difference in height of the crack edges exceed a few decimeters: the settlement and the cracks result from a landslide of the natural slope where the road is located. Transverse cracks and smaller settlements may result from slides of the transition surface between cuts and embankments where well-known construction codes were ignored. But slides and instability in embankment slopes causing longitudinal curved cracks with both ends on one side of the pavement (Figure 6) are most widespread.

Longitudinal Cracks and Settlement Along the Shoulder

Ingress of water in pavements can be restricted but not totally avoided by proper design and good construction. Pervious pavement layers must therefore be provided with adequate drainage. Where drainage is absent, water may seep into pavement pervious layers, usually crushed-stone base courses, and flow to lower sections along the transverse and the longitudinal grades, and may soak the subgrade along the lower shoulder in curves, and both shoulders in straight alignments. This process will cause the subgrade and pavement-bearing capacity to fall. Traffic loads then cause settlements and longitudinal cracks along the shoulders. This association of distress, of course, does not develop along superelevated shoulders.

After rainfall, water may flow out of longitudinal cracks along the shoulders in low sections.

TABLE 1 SUMMARY OF ASSOCIATIONS OF DISTRESS TYPES

Distress type	Diagnosis	Possible remedies
Ravelling, alligator cracking, no distortions	Ageing of bituminous wearing course	Patch., reseal. if low, recycl., overl., reconst. if high degree of severity
Bird bath, alligator cracking	Local low bearing capacity	Replace pav. materials, improve drainage
	Incipient fatigue	Drainage, recycling or overlaying
Wide ruts, alligator cracking	Fatigue granular materials	Drainage, overlaying or recycling
	Consolidation gran. materials	Levelling course
Narrow ruts, heaving (thin wearing course)	Insufficient thickness	Overlaying or reconstruction
	Low grade materials	Recycling, overlaying or reconstruction
Crack bounded settlement	Slope slide or instability	No pavement remedy
Longit. cracks, settl. along low shoulders	Poor pavement subdrainage	Place transv. interceptor and shoulder drains; reconstr. pervious shoulders
Longitudinal crack, settlement with constant width	Underdesigned, poorly compact. widening	Excav. widening, cut pav. at 45 degr., reconst. with adequate thickness., materials and compaction
Eroded shoulder, longitudinal crack	Poor lateral support	Patch pav.and reconstruct shoulder using erosion resistant materials
Block cracking (unif. spacing)	Shrinkage of base material	Seal reflex cracks
Block cracking on wheel paths	Structural failure	Overlaying, recycling or reconstruction
Crescent-shaped cracks, peeling	Weak tack or prime coat	Patching
Peeling in the wheel paths	Constr. defect, weak base	Overlay; recycle; scarify and reconstruct upper pavement layers
Longit. cracks and settlements	Poor embankm. compaction	Levelling course
Longit. cracks, differential settlements, undul.	Expansive soil subgrade	Reconst. and treat subgr. with lime; place imperv. membrane and overlay
Local shearing (depression and heaving)	Low grade pav. materials	Excavate and replace pavement materials
	Water seepage	Excavate, drain and patch

TABLE 1 *(continued on next page)*

TABLE 1 *(continued)*

Corrugations, undulations	Unstable pav. materials	Overlaying; recycling
	Poorly compact. labor intensive construction	Levelling course
Loss of water and fines	Poor subdrain., attrition of pav. materials	Place transv. interceptor and shoulder drains; reconst. perv. shoulders
Blisters, cracking	Salt heaving (arid envir.)	Patch, seal to restrict evaporation; reconstruct with barrier to stop salt migration
Blisters, crack., rust staining	Tropical weathering volcanic aggregates	Patch; recycle or overlay

FIGURE 1 Alligator cracking and raveling without distortion indicating bitumen aging.

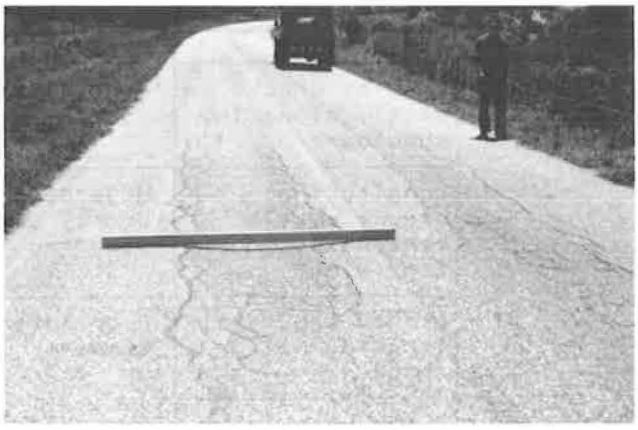

FIGURE 3 Wide ruts and alligator cracking indicating fatigue of a granular pavement.

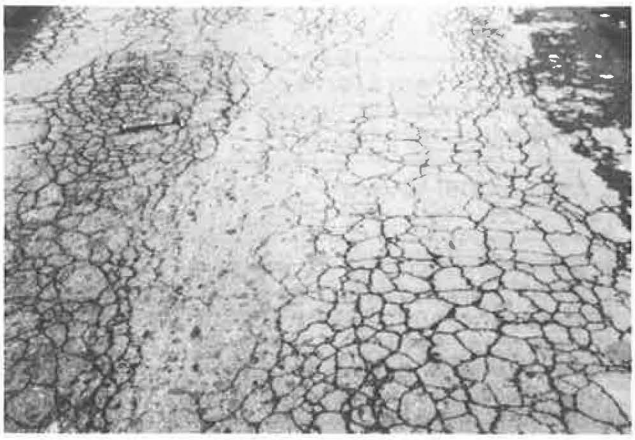

FIGURE 2 Alligator cracking of a depressed area indicating low bearing capacity of pavement structure.

FIGURE 4 Wide ruts resulting from traffic consolidation of a lateritic boulder pavement on a lateritic plateau (National Road 4, Tampoketsa Plateau, Madagascar).

FIGURE 5 Narrow ruts and longitudinal heaving resulting from weathering of a basaltic natural gravel (National Road 5B, near Sambava, Madagascar).

Longitudinal cracks resulting from subdrainage inadequacies are usually irregularly curved and multiple. Underdesigned or poorly compacted partial-lane pavement widenings also cause longitudinal cracks and settlements, but in this case the cracks are straight and parallel to the pavement edge.

Eroded Shoulder and Longitudinal Edge Crack

After shoulder erosion by water and traffic, the pavement edge has no lateral support and traffic loads cause longitudinal cracks with tilting but without settlement. This association of distress may develop on superelevated shoulders.

Block Cracking

The pattern formed by regularly spaced transverse and longitudinal cracks is known as block cracking (Figure 7).

Soil-cement and some other pavement materials develop shrinkage cracks forming a block pattern with dimensions usually in the range of 0.6 to 1.2 m. The initial cracks are transverse cracks starting at the pavement edges. These progress to the pavement center and the longitudinal cracks develop later. Sometimes the whole pavement surface is affected by block cracking. In other cases, cracking does not go beyond the initial transverse cracks along the shoulders.

FIGURE 7 Block cracking of a soil-cement base: Kilometer 89, National Road 6, Madagascar.

Shrinkage block cracking of stiff bases reflects to the wearing course. It is not associated with distortion and only requires crack sealing.

Block cracking resulting from failure of stiff pavement materials under loading has shorter transverse than longitudinal spacing. It may be associated with rutting. It develops along wheel paths and will require pavement strengthening (Figures 8 and 9).

Crescent-Shaped Cracks and Peeling

Peeling of the wearing course occurs following tangential shearing by traffic. It is often associated with crescent-shaped cracks pointing in the direction of traffic flow (Figure 10).

In most cases, peeling indicates some weakness of the prime or tack coat binding the wearing course and the base course,

FIGURE 6 Longitudinal curved crack and settlement resulting from embankment slope slide, National Road 7, in the vicinity of Ambatolampy, Madagascar.

FIGURE 8 Block cracking resulting from tensile failure of a soil-cement pavement: Masiaka Road, Sierra Leone, 1982, before rehabilitation.

FIGURE 9 Two different families of block cracking, Kilometer 0+200, National Road 32, Madagascar: widely spaced shrinkage cracks, and narrow tensile failure block cracks associated with ruts.

but in some cases the level of the shear surface is underneath the prime coat and results from construction defects such as low cement content of soil-cement bases or compaction of thin layers of base material for leveling purposes without previous scarification.

Longitudinal Cracks and Settlements

Traffic compaction of poorly compacted embankments results in longitudinal cracks and settlements with development of a concave cross section (Figure 11).

Undulations, Longitudinal Cracks, and Differential Settlements

Shrinkage of expansive soil subgrades during the dry season causes longitudinal cracks starting at the shoulders and progressing to the axis. Transverse cracks do not develop (Figures 12 and 13).

FIGURE 10 Peeling and crescent-shaped cracks, National Road 1, Zaire: the tack coat binding the soil-cement base to the wearing course was sheared by driving up heavy vehicles.

FIGURE 11 Longitudinal cracks and settlements of a poorly compacted embankment between Malaimbandy and Miandrivazo, National Road 35, Madagascar.

Rainfall during the wet season flows into the cracks causing internal erosion along the deep shrinkage cracks. After some climatic cycles, traffic loading and internal erosion will either cause differential settlements along the longitudinal cracks or collapsed voids (Figures 14 and 15).

Differential swelling along the road results in longitudinal undulations (Figure 16). Transverse settlements are typically found above culverts.

Local Shearing

Local shearing of pavement or subgrade materials takes the form of rounded depressions with some peripheral heaving (Figure 17).

It can either result from local water seepage in the subgrade (natural springs) or from low-grade pavement materials (with

FIGURE 12 Longitudinal cracks developed during the first dry season in sections of National Road 3, Kilometer 73, Benin, where the embankment had been constructed with expansive clays (lama depression). The cracks were carefully sealed but reopened during the next dry season.

FIGURE 13 Well-developed distress pattern typical of expansive subgrade soils; longitudinal cracks and differential settlements (National Road 6, Kilometer 225, Madagascar).

FIGURE 14 Longitudinal cracks and collapsed voids (National Road 6, between Port Bergé and Antsohihy, Madagascar).

FIGURE 15 Longitudinal cracks, settlements, and collapsed voids (National Road 6, between Mampikony and Port Bergé, Madagascar).

FIGURE 16 Longitudinal cracks, settlements, and undulations typical of expansive subgrade soils (National Road 6, between Mampikony and Port Bergé, Madagascar); the car is parked on a stable subgrade section.

high local clay content or friable gravel fraction). Figure 17 shows a local shearing of a friable lateritic sandstone base.

Corrugations

Corrugations of bitumen-surfaced roads usually result from oscillating traffic loading of unstable bitumen mixes, soft aggregates, or soft-penetration-grade bitumen (Figure 18).

Irregular undulations are typical of pavements constructed by labor-intensive methods, particularly telford and macadam pavements, and result from traffic compaction.

Loss of Water and Fines

Water and fines ejected through cracks indicate poor pavement subdrainage. Water ingress may cause saturation of pavement layers. Traffic loading may result in high pore water pressure and create conditions for accelerated attrition of pavement materials and ejection of fines.

FIGURE 17 Local shearing of a soft sandstone gravel, Kilometer 3 from Pobe, National Road 3, between Porto Novo and Pobe, Benin, 1985, before rehabilitation.

FIGURE 18 Corrugations at Kilometer 20, National Road 7, Madagascar.

FIGURE 19 Cracked blisters and rust staining along the shoulders of National Road 12, Kilometer 133, between Manakara and Farafangana, Madagascar.

Blisters and Cracking

Under arid climates, blisters result from salt heaving (*6*). They are not necessarily associated with cracks, although evaporation of pavement moisture through cracks may cause blistering along these.

Blisters developing along the shoulders of National Road 12, Farafangana, Madagascar, were found to be associated with small radiating cracks and rust staining (Figure 19). They resulted from tropical weathering of volcanic rock aggregates, subsequent migration of dissolved iron, and expansion of residual clay minerals being formed within the aggregates of the hot-mix wearing course.

CONCLUSIONS

The assessment of bitumen-surfaced road pavements before maintenance or rehabilitation works should be based on detailed condition surveys and well-founded engineering judgment to achieve good-quality and low-cost goals.

The basis for sound engineering judgment is careful observation of distress types, with an objective rating of the extent and degree of severity and interpretation of distress associations.

Individual distress types may have different possible causes but distress associations can provide clues to proper diagnosis of the causes of pavement distress and to the required maintenance and rehabilitation work.

Thus, for instance, the association of raveling and cracking on an undistorted pavement results from bitumen aging and indicates that a resealing is required, whereas wide ruts and alligator cracking may result from fatigue of granular pavements and indicate the end of the structural design life and the need for strengthening. The association of cracks and settlements along the lower shoulders unequivocally indicates the lack of proper pavement subdrainage.

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Road Network Stabilization Program in Ghana

KWESI ABBEY SAM

Over the past two to three decades, road networks in most African countries have expanded more extensively than the respective maintenance budgets and institutional capacities. Consequently, the level of maintenance of the roads has been low. During the same period, there was excessive overloading of the road pavements with axle loads far above pavement design capacities. As a result, large sections of the networks deteriorated rapidly. The experience in Ghana is similar to, or worse than, that of many other African countries. By the beginning of 1988, 30 percent of the trunk (main) roads were in good condition, with 30 percent in fair condition, and 40 percent in poor condition. Expenditure on road maintenance, though substantial relative to the overall national budget, was nevertheless not yielding expected returns. Something had to be done quickly to arrest the situation and save the road network and the economy from imminent collapse. It was postulated that to minimize total transport costs, including expenditures to rehabilitate the roads, a condition mix for the network, as a whole, of 70 percent good, 20 percent fair, and 10 percent poor would be optimal. Taking into consideration budgetary constraints, a practical program for clearing maintenance backlog and stabilizing the network at the optimal condition mix was drawn up for execution in 7 years. The annual budget required to execute the program was found to be two to three times that of the budget at that time. In addition to the introduction by the government of Ghana of the road fund to increase the road maintenance budget, assistance was sought from the World Bank for the execution of the program. Individual road sections to be rehabilitated under the program were selected through engineering and economic studies conducted by local consultants in association with some foreign consultants. Work is being executed through local and international competitive bidding. In spite of initial delay in the start-up of the program, the project is proving to be highly successful.

Road deterioration is a worldwide phenomenon. All new roads, whatever the method of construction or the materials used, deteriorate with time and use. It is the actions taken to contain the situation that make the phenomenon either insignificant or alarming.

A study in 85 developing countries that received World Bank assistance for roads indicated that these countries had a rapid expansion of their road networks during the 1960s and 1970s. Unfortunately, maintenance budgets and institutional capacities did not increase at the same rate. As a result, large road networks, built at great expense, were not adequately maintained (*1*). The situation worsened as vehicles with axle loads far exceeding design capacity used the roads. The combined effect of all these is the rapid deterioration the roads have gone through.

It is estimated that about $90 billion will be required over a 10-year period to reconstruct the failed sections of the roads, to strengthen existing pavements that have been stressed to the limit, and to execute routine and periodic maintenance activities on these roads (*1*). It is predicted that if nothing is done to improve the management of these roads, the eventual repair bill could easily increase two- to three-fold.

The situation in countries in Africa is similar to, or worse than, that of other developing countries elsewhere. Over the last two to three decades, there has been a rapid construction or reconstruction of sections of the road network. By 1985, sub-Saharan Africa had about 130,000 km of paved roads, 355,000 km of gravel roads, and 425,000 km of earth roads and tracks (*2*). For reasons already stated, it has not been possible to keep up the right level of maintenance. As a result, most of these countries have large portions of their networks (usually exceeding 45 percent) in poor condition. The related economic loss to each of these nations is astronomical.

About $15.0 billion will be required to reinstate sections of the network that have deteriorated beyond economic repairs. A further $1.8 billion is required annually for routine and periodic maintenance and pavement strengthening (*2*). Most of these nations can ill afford these heavy expenditures from their dwindling revenue from the exports of their commodities the prices of which keep falling on the world market. A way must be found out of the dilemma to prevent a total collapse of the road transport system, and therefore the economy.

Ghana is no exception. The trunk roads in Ghana consist of 5 782 km of paved roads and 8 352 km of gravel roads, giving a total of 14 134 km. Most sections of the trunk road network were built or reconstructed in the period from the early 1950s to the late 1970s, but have suffered from serious neglect of maintenance subsequently. By the beginning of 1988, about 40 percent of the trunk roads were in poor condition, 30 percent in fair condition, and only 30 percent in good condition. Because of the inadequate maintenance, a great deal of backlog of road rehabilitation has built up that urgently requires to be cleared. In addition, normal routine and periodic maintenance should be executed annually. For several years, funds available for periodic maintenance covered only 20 percent of the estimated needs. A total collapse of road transport is imminent if this trend is allowed to continue.

A collapsed road transport system would inevitably lead to a collapsed economy and this has not been the vision of this nation. It is the vision and hope of the government and the people that Ghanaians would ride on reasonably smooth and safe roads. To achieve this vision, the backlog of maintenance must be cleared and a reasonable level of routine and periodic maintenance must be sustained. This cannot be achieved in 1 year. It has to take place over a reasonable period of time

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and yet not spread out over so long a period as to make it ineffective. The Road Network Stabilization Program is a practical approach towards achieving this vision. The program for clearing the backlog of maintenance until stabilization is reached is described. It is hoped that this will be achieved in 7 years, when normal routine and periodic maintenance will be sustained to keep the network at a reasonable level of service.

ROAD NETWORK IN GHANA

Background

Ghana, a former British Colony (Gold Coast), is located in West Africa between longitudes 1°12′ E and 3°15′ W and latitudes 4°45′ N and 11°11′ N. The nation became independent in 1957 and a republic in 1960. It has an area of 239,000 km^2 and a population of about 13 million (1984) with an annual population growth rate of 2.6 percent. The climate is tropical with temperatures normally ranging between 18°C and 35°C. It has a rolling terrain with hilly areas rising to a maximum of about 880 m. The terrain is generally flat in the south around the capital, Accra, and in the northern part of the country. Annual rainfall varies from about 710 mm around Accra to about 2,180 mm in the western region. Located in the tropical forest zone, farming forms the greatest vocation of the people. Cocoa is the leading export crop; timber follows in third position after minerals. In recent times, the production of grain and pineapple has been on the increase. As its former name (Gold Coast) suggests, Ghana is rich in gold. Other minerals mined include manganese, bauxite, and diamonds. Exploration of crude oil continues and it is hoped that fields yielding commercial quantities will soon be found. Apart from the smelting of alumina to aluminum, the nation's industry is in infancy. Ghana, however, has a great potential for industrial development.

Development of the Road Network in Ghana

Before 1910, practically no motorable roads existed in the country and there was little need for them. Road construction was started seriously at the beginning of the second decade together with the systematic reconstruction, in concrete and steel, of old timber bridges and culverts on the main roads.

The rapid development of roads in the 1920s was slowed down during the great depression of the 1930s. The pace of road construction was quickened again from 1949. At this time, the Public Works Department (PWD) maintained 1,053 km of bituminous-surfaced roads and 3,332 km of gravel roads. This rapid development continued in the 1950s, 1960s, and 1970s. Table 1 shows the growth of the road network in Ghana.

Condition of Present Road Network

The approximately 14,000 km of trunk roads in the country have been classified functionally as primary, major secondary, and minor secondary roads. The classification is not based on traffic but the long-term function the road is expected to perform. Primary roads link up regional capitals, major population centers, and Ghana itself to neighboring countries. They are also corridors for large movement and long-distance journeys. Secondary roads, on the other hand, serve as collectors between the feeder roads and the primary system and provide regional cohesion. Table 2 presents the lengths of roads in the various classes.

TABLE 1 GROWTH OF THE ROAD NETWORK IN GHANA

YEAR	LENGTH OF TRUNK ROADS - KM		
	GRAVEL	PAVED	TOTAL
1949	3,332	1,053	4,385
1953	3,946	1,760	5,706
1955	3,979	2,230	6,209
1960	4,013	3,059	7,072
1965	5,408	3,512	8,920
1970	6,620	4,258	10,878
1975	7,538	4,838	12,376
1980	7,753	5,419	13,172
1985	8,053	5,601	13,654
1989	8,352	5,782	14,134

TABLE 2 LENGTH OF ROADS BY FUNCTIONAL CLASSIFICATION

ROAD CLASS	LENGTH	AVERAGE DAILY TRAFFIC
Primary	3,998 (28%)	100 - 4,000
Major Secondary	4,181 (30%)	100 - 1,000
Minor Secondary	5,955 (42%)	50 - 300

In 1960/1961, resources for road maintenance and rehabilitation were greatly reduced. This, obviously, resulted in reduced maintenance. The situation was worsened when large semitrailer trucks were allowed into the country towards the latter part of the 1960s. Roads that had been deteriorating slowly moved at a quickened pace on the path of deterioration. Even though the desire to solve the problem was high, as far back as the mid-1970s, not much could be done without adequate funding. By the end of 1987, 40 percent of the trunk road network was in poor condition, 30 percent in fair condition, and only 30 percent in good condition. Although the total length of paved roads did not change much, the length of sections with high-level pavement (portland cement and asphaltic concrete) increased more than three-fold from about 150 km in 1985 to about 588 km in 1989. This increase is the result of the reconstruction of roads with high traffic volumes.

Traffic Levels

The levels of traffic on the trunk road network in Ghana are currently not high. As presented in Table 3, the average ADT is less than 1,000 vpd over 76 percent of the paved roads and less than 200 vpd over 78 percent of the gravel-surfaced roads. On most sections, the medium- and heavy-truck traffic accounts for less than 30 percent of total volume and has equivalent standard axle load factors that range from about 1.5 to 3.0. These do not seem exceptionally high in comparison to those in other sub-Saharan countries. However, measurements over the past 2 to 3 years on recently reconstructed roads indicate some excessive overloading of the pavements. Also, the traffic growth on some of these reconstructed roads is far in excess of predicted growth factors.

Pavement Strength

The road pavement in most cases consists of 150 mm of gravel subbase, 150 to 200 mm of gravel base, followed by either a single or double surface dressing. In a few cases, the surfacing is either portland cement concrete or asphaltic concrete. The subgrade strength, as measured by the California bearing ratio (CBR) ranges from 5 to 15 percent on average. These values are fairly suitable for normal pavement construction. In most cases, the pavement structural number (SN) varies from 1.5 to 3.0. The design for gravel roads calls for 150 to 200 mm of well-compacted, good-quality gravel surface. Yet, in many cases this has not been replenished in a long time.

Road Deterioration Cycle

Studies (*1*) conducted in several parts of the world show that new paved roads deteriorate slowly and almost imperceptibly during their first 5 to 10 years, if inadequately maintained. This phase is followed by a phase of increasing deterioration as the road condition changes from good to fair. Without timely intervention, the rate of deterioration becomes even faster, leading to structural failure of the pavement. At this stage, the road is in poor condition. This relationship can be shown as the roughness of the road against time. Gravel roads that are not maintained deteriorate more rapidly and at a fairly uniform rate throughout their life cycle.

In Ghana, it is estimated that with basic routine maintenance including patching of potholes and timely repair of cracks and edge failures, a newly constructed paved road can remain in good condition for about 10 years; thereafter, in fair condition for another 7 years. While it is in fair condition, it can be restored to good condition with a thin overlay or a surface reseal. If no periodic maintenance is performed during this time, the pavement will deteriorate into poor condition. At this time, a total reconstruction or resurfacing of the pavement will be required to restore the road to good condition (see Figure 1). On this basis, and assuming uniform age distribution of the roads, it is conjectured that about 1/10th of the paved roads in good condition will make the transition to fair condition each year and about 1/7th of the roads in fair condition will move to poor condition during the same time. For gravel roads, the corresponding fractions are both assumed to be 1/7th.

TABLE 3 TRAFFIC DISTRIBUTION OVER THE TRUNK ROAD NETWORK

ADT RANGE (vpd)	PAVED ROADS		GRAVEL ROADS	
	KM	%	KM	%
> 3000	142	3	-	-
1000 - 3000	1,207	21	58	1
500 - 1000	1,288	22	380	4
200 - 500	2,039	35	1,441	17
50 - 200	1,030	18	3,409	41
< 50	76	1	3,064	37
TOTAL	5,782	100	8,352	100

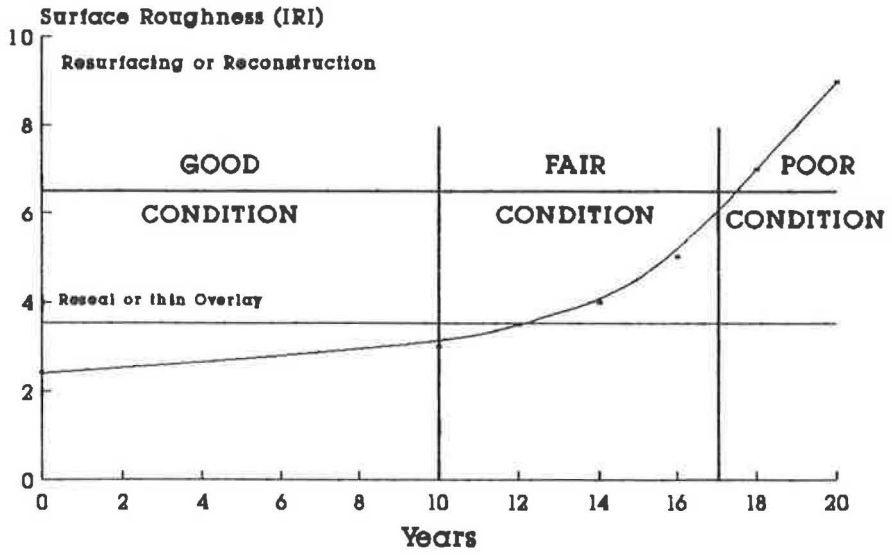

FIGURE 1 Road deterioration phases.

Road Maintenance and Rehabilitation Options

In order to arrest road deterioration, some maintenance and rehabilitation activities have to be carried out. It is assumed that basic routine maintenance will be executed in each case and does not form part of the options. Paved roads in fair condition are normally resealed or a thin overlay applied to bring them from fair to good condition. The estimated cost of the resealing is $22,000 per km. (Incidentally, application of thin overlays on previously surface-dressed roads has presented some problems in recent times in Ghana and is not much favored.)

Roads in poor condition are restored to good condition either through resurfacing or major rehabilitation. In resurfacing, the existing surface is scarified, a new base is laid, and it is then surface dressed. In most cases, sources of failure can be attributed to poor drainage. Some drainage reconstruction, raising or rebuilding of embankment, and minor shoulder repairs are often included in resurfacing operations. The estimated cost for resurfacing varies from $35,000 to $65,000 per kilometer. For the network stabilization program, $50,000 per kilometer was assumed. Where the paved road is badly deteriorated, major rehabilitation or reconstruction is carried out. Activities include major drainage repairs, realignment, and minor earthworks and the construction of new pavements or reconstruction of the pavement. The estimated cost for the major rehabilitation is $250,000 per kilometer.

ROAD REHABILITATION PLAN FOR NETWORK STABILIZATION

Basic Principles

Studies conducted by the World Bank in Kenya, Brazil, and India, and from which the Highway Design and Maintenance Standards Model (HDM) and the Expenditure Budgeting Model were developed, indicate clearly that transportation cost can be minimized if total transport cost of the network (construction cost + maintenance cost + vehicle operating cost) is minimized (*1*). In a road network, this objective is achieved by keeping the roads at an optimal mix of the different conditions of the roads in the entire network.

Even if funds are unlimited, the optimal program is not necessarily to keep 100 percent of the network in good condition at all times. Past studies in other countries (*3*) have shown, for example, that periodic maintenance on the very low-volume paved roads [the average daily traffic (ADT) less than about 200 vehicles per day (vpd)] is difficult to justify economically, particularly under budget constraints. Such roads warrant only routine maintenance including patching of potholes and sealing of cracks until such time as the traffic increases sufficiently to justify pavement reconstruction.

Assumed Optimal Mix

When the rehabilitation plan was being developed in Ghana, not enough data were available to determine the optimal mix for the trunk road network, even though the HDM was available. In the interim, a decision was taken to have a desirable mix consisting of about 9 600 km in good condition, 3 000 km in fair condition, and 1 500 km in poor condition. A breakdown by surface type is presented in Table 4.

Assuming it is planned to stabilize the network to the desirable condition mix presented in Table 4 within a given number of years, the target condition mix to be achieved at the end of each year can be determined by spreading the difference between the existing condition and the desired condition equally over the plan period. The actual length of roads that qualify for periodic maintenance or rehabilitation can then be worked out in each category by taking the difference between the predicted condition mix at the end of the year without maintenance and the target mix to be attained in that year.

TABLE 4 DESIRABLE CONDITION MIX OF THE TRUNK ROAD NETWORK

ROAD TYPE	UNIT	GOOD	FAIR	POOR	TOTAL
Paved	KM	4,282	1,000	500	5,782
	%	74	17	9	100
Gravel	KM	5,352	2,000	1,000	8,352
	%	64	24	12	100
Total Network	KM	9,634	3,000	1,500	14,134
	%	68	21	11	100

Seven-Year Stabilization Program

The selection of the length of period for the stabilization program was dictated primarily by the availability of funds to carry out the program and also by the development of the institutional arrangements for the implementation of the program. It came out that the optimal stabilization period would be 7 years for the paved roads and 5 years for the gravel roads. Table 5 presents the 7-year stabilization program derived in the manner described, except that to fit the physical output to the budget requirements, to build up the major rehabilitation requirements, and to build up the major rehabilitation program slowly over the 7-year period, it was necessary to alter the intermediate target lengths in some years.

Under the plan shown, the gravel road network will stabilize in 5 years. The periodic maintenance needs on the paved network will stabilize in 7 years, during which time the backlog requiring major rehabilitation would also be cleared. As indicated in Table 5, the program requires 918 km of regraveling in 1988, increasing to 1,224 km in 1992, and stabilizing thereafter to 765 km per annum. As more and more roads are added into the good-condition category, resealing needs will increase steadily from 50 km in 1988 to 285 km in 1995 and thereafter, whereas resurfacing needs will decrease from 409 km in 1988 to 83 km in 1995 and thereafter. Major rehabilitation and strengthening of the badly deteriorated roads is planned to increase from 150 km in 1988 to a high of 270 km by 1992 before decreasing to 222 km in 1994 and 60 km annually thereafter (Table 6).

The periodic maintenance expenditure in the first 5 years amounts to $28 million per annum, decreasing to $18.8 million over the next 2 years, before stabilizing at $15.8 million per annum thereafter. Funds required for major rehabilitation increase gradually from $37.5 million in 1988 to $67.5 million in 1992, falling off to $55.5 million in 1994, and to $15 million per annum thereafter.

The portion of the combined network that is in good condition increases steadily from about 30 percent to about 70 percent at the end of the plan period and is kept at that level with funds that are well within the existing maintenance budget. During the same period, the portion of roads in poor condition decreases from about 40 percent to about 10 percent, and the portion in fair condition decreases from 30 to about 20 percent. Figure 2 shows graphically the projected trunk road network condition on the assumptions that the expected maintenance and rehabilitation works under the program are carried out fully each year and that no significant expansion of the network takes place through new construction during this time.

With increased budgetary allocations, the backlog of road maintenance could be cleared and the network stabilized in 5 years (Table 7). Even though there is the advantage of the network stabilizing in a shorter time, the nation would be saddled with heavy expenditures during the first 3 years (additional $39 million compared to the 7-year program).

The 7-year option builds up gradually over the first 3 years at budgetary levels within the reach of the government of Ghana and projected World Bank [International Development Association (IDA)] credit. One other advantage is that the institutional and construction capacities would have built up gradually to a maximum by 1992, using both local and external contractors, with the latter being phased out by the time the network is stabilized.

Though the 10-year program would require even less annual outlays, yet considering the delay in economic returns for the investment this may not be the optimal economic solution. The expenditure profiles for the three options are shown in Figure 3.

Stabilization Program with Inadequate Funds

If funds are not made available to embark on the accelerated program of clearing the backlog as stipulated, then it can be shown that with only the current committed level of funding (approximately $16 million for periodic maintenance and $25 million for major rehabilitation), it would take about 20 to 30 years to accomplish the same task of stabilizing the network to the desired condition mix. This effect is shown in Figure 4.

IMPLEMENTATION OF THE PROGRAM

The first phase of the Road Network Stabilization Program is being implemented by the Ghana government with World Bank (IDA) support under a Transport Rehabilitation Project (TRP–1). The project, originally planned for execution in 1988 and 1989, is likely to extend into 1991.

TABLE 5 NETWORK STABILIZATION PLAN (1988 THROUGH 1995) AND PROJECTED ROAD CONDITIONS

YEAR	ITEM	PAVED (5782 km)			MAJOR REHAB	GRAVEL (8352 km)			TOTAL COST (mn $)	
		GOOD	FAIR	POOR		GOOD	FAIR	POOR	PER.MTC.	MAJ.REH.
1988	Starting Position	1,593	1,236	2,953		2,672	2,989	2,691		
	Percent Distribution	28	21	51		32	36	32		
1989	Without 1988 M&R Prog.	1,434	1,219	3,130		2,290	2,944	3,118		
1988	Expected M&R Prog.	609	-50	-409	-150	918	-153	0	28.0	37.5
1989	With 1988 M&R Prog.	2,043	1,169	2,571		3,208	2,791	3,118		
	Percent Distribution	35	20	44		35	31	34		
1990	Without 1989 M&R Prog.	1,839	1,206	2,738		2,750	2,851	2,752		
1989	Expected M&R Prog.	644	-104	-375	-165	994	-257	-737	28.0	41.3
1990	With 1989 M&R Prog.	2,483	1,102	2,198		3,744	2,594	2,015		
	Percent Distribution	43	19	38		45	31	24		
1991	Without 1990 M&R Prog.	2,234	1,193	2,355		3,209	2,758	2,385		
1990	Expected M&R Prog.	680	-150	-345	-185	1,071	-361	-710	28.0	46.3
1991	With 1990 M&R Prog.	2,914	1,043	1,825		4,280	2,397	1,675		
	Percent Distribution	50	18	32		51	29	20		
1992	Without 1991 M&R Prog.	2,623	1,186	1,974		3,668	2,666	2,018		
1991	Expected M&R Prog.	710	-160	-330	-220	1,147	-467	-680	28.0	55.0
1992	With 1991 M&R Prog.	3,333	1,026	1,424		4,815	2,199	1,338		
	Percent Distribution	58	18	25		58	26	16		
1993	Without 1992 M&R Prog.	3,000	1,212	1,571		4,128	2,573	1,652		
1992	Expected M&R Prog.	777	-212	-295	-270	1,224	-572	-652	28.0	67.5
1993	With 1992 M&R Prog.	3,777	1,000	1,006		5,352	2,001	1,000		
	Percent Distribution	65	17	17		64	24	12		
1994	Without 1993 M&R Prog.	3,399	1,235	1,149		4,587	2,479	1,286		
1983	Expected M&R Prog.	650	-235	-165	-250	765	-479	-286	18.8	62.5
1994	With 1993 M&R Prog.	4,049	1,000	734		5,352	2,000	1,000		
	Percent Distribution	70	17	13		64	24	12		
1995	Without 1994 M&R Prog.	3,644	1,262	876		4,587	2,479	1,286		
1994	Expected M&R Prog.	637	-262	-153	-222	765	-479	-286	18.8	55.5
1995	With 1994 M&R Prog.	4,281	1,000	501		5,352	2,000	1,000		
	Percent Distribution	74	17	9		64	24	12		
1996	Without 1989 M&R Prog.	3,853	1,285	644		4,587	2,479	1,286		
1995	Expected M&R Prog.	428	-285	-83	-60	765	-479	-286	15.8	15.0
1996	With 1989 M&R Prog.	4,281	1,000	501		5,352	2,000	1,000		
	Percent Distribution	74	17	9		64	24	12		
1996	Expected M&R Prog.	428	-285	-83	-60	765	-479	-286	15.8	15.0

M&R - Maintenance and Rehabilitation

Engineering Study and Design

Four local consultants in association with foreign firms were engaged to study the road network and to determine the nature of periodic maintenance required on the various sections of the network and to order the anticipated projects by priority. The country was divided into four geographic zones and each consultant was assigned one zone to study. Table 8 presents the road lengths studied.

The data gathered included information on road characteristics, pavement condition, properties of materials, traffic volumes and distribution, etc. The major component of economic benefits was taken as the reduction of vehicle operating costs (VOCs). The World Bank's vehicle operating cost submodel was used to derive the VOCs as a function of road roughness and topography for seven different types of vehicle in Ghana. These were then supplied to all four consultants for use in their economic analysis.

From data collected, maintenance activities to be executed were determined and plotted on line diagrams. Work quantities were estimated for each type of activity and extended to cost.

TABLE 6 SEVEN-YEAR NETWORK STABILIZATION PLAN (km)

YEAR	RESEALING	RESURFACING	REGRAVELLING	TOTAL PERIODIC MTCE	MAJOR REHAB	TOTAL COST	
						PER. MTCE	MAJ. REH.
1988	50	409	918	1,377	150	28.0	37.5
1989	104	375	994	1,473	165	28.0	41.3
1990	150	345	1,071	1,566	188	28.0	46.3
1991	160	330	1,147	1,637	220	28.0	55.0
1992	212	295	1,224	1,731	270	28.0	67.5
1993	235	165	765	1,165	250	18.8	62.5
1994	262	153	765	1,180	222	18.8	55.5
1995	285	83	765	1,133	60	15.8	15.0
1988-89 TRP-1	154	784	1,912	2,850	315	56.0	78.8

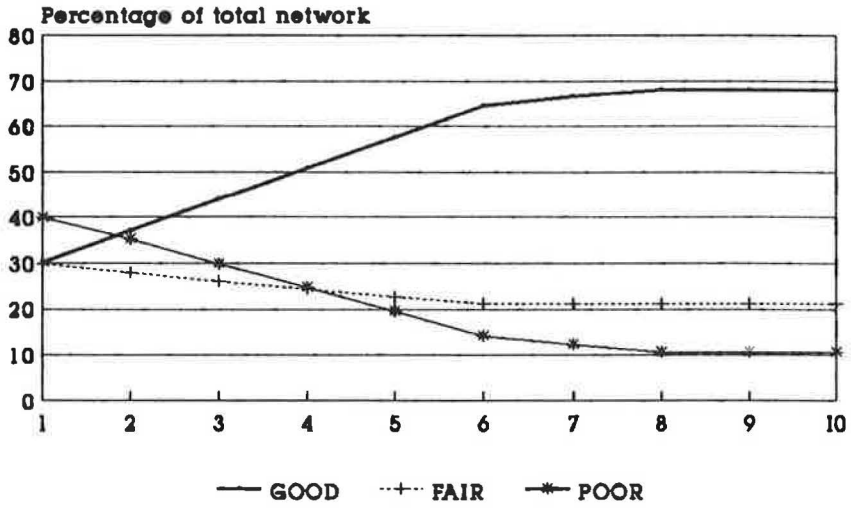

FIGURE 2 Projected network condition with accelerated stabilization program.

Priority Rating of the Works

From the Benefit and Cost streams, the Net Present Value (NPV) and Economic Rate of Return (ERR) were determined for each road section. Table 9 presents the result of the economic analysis for Eastern Region in Zone 4. High priority was assigned to projects with high ERR values, with 15 percent as the cut-off point.

Executing the Program

Tender documents were prepared in packages suitable for local competitive bidding (LCB) as well as international competitive bidding (ICB). The LCBs were valued at less than about $0.5 million each, whereas the ICBs were valued at around $5.0 million each. For TRP–1, there were three phases of LCB and one phase of ICB. Tables 10 and 11 show packaging for LCB Phase 2 and ICB Phase 1, respectively, for Zone 4.

LCB Phase 1 projects were put to tender in June 1988 and awarded for execution towards the end of 1988. By the end of 1989, most of the works had been completed. LCB Phase 2 projects were awarded toward the middle of 1989 and are expected to be completed in 1990. With the scarcity of funds, LCB Phase 3 projects were delayed. A portion of the Phase 3 projects was being awarded in early 1990 and the remainder will be awarded towards the end of 1990 for execution in

TABLE 7 COMPARISON OF NETWORK STABILIZATION PLANS (COMBINED PERIODIC MAINTENANCE AND MAJOR REHABILITATION)

YEAR	FIVE YEAR PLAN		SEVEN YEAR PLAN		TEN YEAR PLAN	
	LENGTH km	ANNUAL EXPENDITURE Mn$	LENGTH km	ANNUAL EXPENDITURE Mn$	LENGTH km	ANNUAL EXPENDITURE Mn$
1988	1,700	73	1,527	66	1,320	42
1989	1,840	80	1,638	69	1,410	50
1990	1,971	95	1,754	74	1,460	56
1991	1,820	83	1,857	83	1,510	59
1992	2,097	83	2,001	96	1,555	61
1993	1,193	31	1,415	81	1,620	67
1994	1,193	31	1,402	74	1,605	69
1995	1,193	31	1,193	31	1,480	64
1996	1,193	31	1,193	31	1,323	44
1997	1,193	31	1,193	31	1,243	35
1998	1,193	31	1,193	31	1,193	31

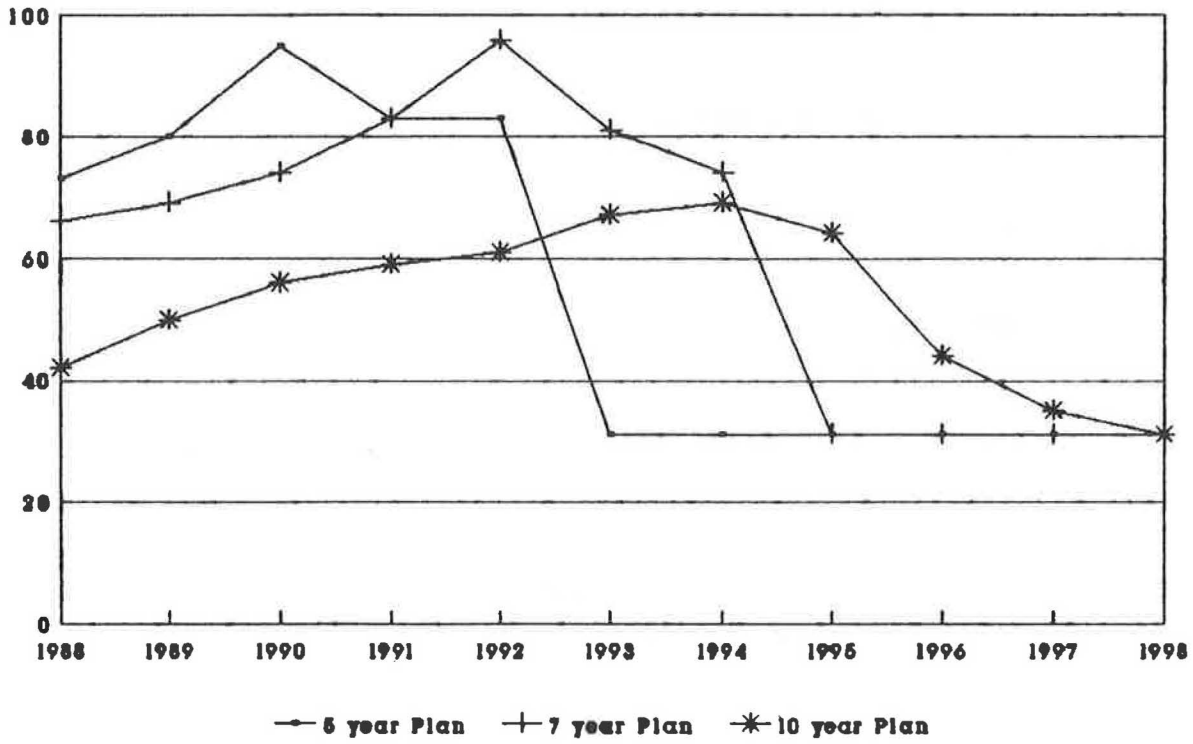

FIGURE 3 Comparison of annual expenditure for alternative programs.

1991. ICB projects were put to tender in 1989 and are undergoing various stages of approval. Work expected to start from January 1990 may now start from July 1990 for completion by the end of 1991.

Funding of the Program

As derived, the program called for almost two- to three-fold increase in the annual budget allocation for periodic maintenance of trunk roads made to Ghana Highway Authority in 1986 and 1987. With the first 2-year slice of the program estimated at $56.0 million, external funding was required. Under the Transport Rehabilitation Project (TRP-1), IDA pledged $26.2 million for the program. Some cofinancing expected under the project did not materialize. Ghana Government is therefore financing the balance from a road fund and the consolidated fund of the central budget.

As a result of this and other factors, the project could not fully start in 1988 as projected. Secondly, it is being spread over 3 years as against the original 2-year period. Table 12 shows the expenditure plan for the 3-year period.

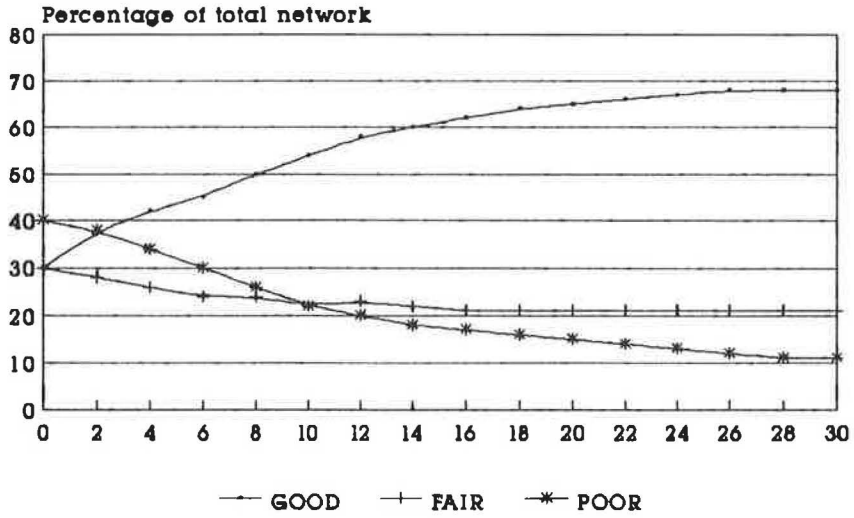

FIGURE 4 Projected network condition without accelerated stabilization program.

TABLE 8 LENGTHS OF ROADS ASSIGNED TO CONSULTANTS FOR STUDY

ZONE	REGIONS	LENGTH OF ROADS STUDIED		
		PAVED	GRAVEL	TOTAL
1	Northern Upper East Upper West	222	1,061	1,283
2	Ashanti Brong Ahafo	509	605	1,114
3	Central Western	356	616	972
4	Eastern Greater Accra Volta	609	362	971
TOTAL		1,696	2,644	4,340

The government of Ghana intends to double proceeds from the road fund in 1990 to ensure that the execution of the program does not deviate completely from the plan.

Technical Assistance

Because clearance of the backlog represented a significant increase in the workload handled by the Ghana Highway Authority, the World Bank–aided project included a technical assistance component that provided key personnel in the areas of project management, planning, and maintenance supervision. With the training of counterparts in these areas and with the provision of logistics support, the respective units of the Ghana Highway Authority are being strengthened, such that the Authority would be able to plan and execute successfully similar and complementary projects in the future. Under the same project, advisory, training, and financial services are being provided also to the local contractors, to enable them to bid competitively and to procure necessary construction equipment. The planned program for 1988/1989 is expected to be achieved by the middle of 1991, at which time plans will also be finalized for implementation of the next 2 or 3 years' program under a Transport Rehabilitation Project, Phase 2 (TRP–2).

Two Years of Network Stabilization Program

Even though the Network Stabilization Program did not take off fully, as expected in 1988, some periodic maintenance works were executed by local contractors and the Mobile Maintenance Unit of Ghana Highway Authority. Totals of 660 km of regraveling, 93 km of resealing, and 82 km of

TABLE 9 TRUNK ROAD MAINTENANCE STUDY FOR EASTERN REGION ROAD NETWORK STABILIZATION: GENERAL SUMMARY, ZONE 4, EASTERN REGION

NO.	ROAD NAME	Surf Type	Length (km)	1988 ADT	Gen ADT	Cond Rating	Proposed Mtce (km) Regrav	Reseal	Resurf	Misc	Est Cost (m¢)	Cost/ km (m¢)	NPV @ 15%	NPV Cost	ERR %
	EASTERN														
5	Koforidua-Asesewa	P	45.9	378	-	V. Good		4.3	41.6		260.6	5.7	934.0	3.6	69
6	Nkurankan-Adukrom-Tron	P	36.5	488	48	V. Good		20.7	13.8		122.4	3.4	849.8	6.9	175
7	Tron-Somanya-Kpong	P	16.1	1503	-	V. Good		1.8	14.3		80.9	5.0	1309.8	16.2	391
8	Manfe-Akropong-Adukrom	P	10.9	567	-	Fair		8.9	2.0		67.1	6.2	69.2	1.0	32
9	Koforidua-Adawso-Manfe	P	32.1	752	-	Fair		10.8	21.3		217.0	6.8	718.9	3.3	56
10	Adoni-Asikuma	P	26.3	999	-	Poor		22.8	3.5		90.1	3.4	782.8	8.7	160
11	Adoni-Akosombo	P	9.5	838	-	Fair			9.5		67.6	7.1	227.7	3.4	56
12	Somanya-Akuse-Asutuare	P	24.0	663	-	Poor		8.6	15.4		64.3	2.7	957.7	14.9	227
13	Tema-Kpong-Adoni	P	65.4	1942	-	Fair		54.3	11.1		229.9	3.5	3541.3	15.4	170
14	Aburi-Dodowa	P	19.9	1499	-	Fair		12.6	7.3		84.8	4.3	519.9	6.1	107
15	New Tafo-Bunso	P	18.2	324	164	V. Poor		1.2	17.0		119.9	6.6	395.6	3.3	89
16	Adeiso-Asamankese	P	25.3	301	76	Poor			25.3		198.9	7.9	268.9	1.4	35
17	Asamankese-Kede	P	40.0	332	83	Poor		2.7	37.3		231.7	5.8	589.1	2.5	67
18	Manso-Osenase	P	19.5	178	45	Poor			19.5		98.9	5.1	157.8	1.6	39
19	Akim Oda-Manso	P	11.4	1624	-	Fair		9.1	2.3		59.4	5.2	423.4	7.1	95
20	Akim Oda-Achiase-Amanfopong	P	13.5	502	-	Fair		5.9	7.6		100.0	7.4	695.5	7.0	113
21	Akim Oda-Ofoase-Ntornang (a)	P	15.5	483	-	Poor			15.5		86.2	5.6	339.7	3.9	98
22	Akim Oda-Ofoase-Ntornang (b)	U	46.2	50	13	Poor	46.2				108.2	2.3	9.6	0.1	17
23	Nkawkaw-Ntronang	U	26.6	89	23	Poor	26.6				55.0	2.1	81.1	1.5	38
24	Anyinam-Abodom	U	43.3	79	20	Poor	43.3				103.0	2.4	91.6	0.9	31
25	Kwaben-Abonaso-Asuom	U	19.8	51	14	V. Poor	19.8				48.0	2.4	10.6	0.2	20
26	Kade-Asuom	U	19.1	102	-	Fair	19.1				37.1	1.9	68.4	1.8	41
	Regional Sub-total		**585.0**				**155.8**	**163.7**	**264.3**	**0.0**					

TABLE 10 GHANA HIGHWAY AUTHORITY, PLANNING DIVISION, TRUNK ROAD MAINTENANCE BY CONTRACT—SUMMARY OF LCB PHASE 2, ZONE 4, JUNE 30, 1989

NO.	ROAD NAME/(Section)	Lot Nos	Surf Type	Length (km)	Proposed Mtce (km) Regrav	Reseal	Resurf	Misc	Est Cost (m¢)	Remarks
	GREATER ACCRA									
9	Sege-Mepe	1,2	U	27.0	13.8		11.2	2.0	96.3	Awarded
10	Kaseh-Ada (10-21)	3	P	11.4		11.4			62.8	Awarded
11	Doryumu-Ayikuma (8-12)	4	P	3.6		3.6			11.4	Awarded
	Regional Sub-total			**42.0**	**13.8**	**15.0**	**11.2**	**2.0**	**170.5**	
	EASTERN									
12	Anyinam-Abodom	1	U	43.3	43.3				100.5	40% Complete
13	Asamankese-Kade	2,3,4	P	40.0		2.7	37.3		300.9	10% complete
14	Adeiso-Asamankase	5,6	P	25.3			25.3		206.8	22% complete
	Regional Sub-total			**108.6**	**43.3**	**2.7**	**62.6**	**0.0**	**608.2**	
	VOLTA									
15	Kpando-Tokor	1	U	4.5			0.6	3.9	47.4	Awarded
16	Dabala-Srogbe	2	P	8.4		6.2	2.2		149.5	Negotiation
17	Kpando-Gbefi-Golokwati	3	U	6.8		1.3		5.5	70.0	Awarded
18	Akatsi-Wute-Ziope	4	U	28.3	28.3				65.4	Awarded
	Regional Sub-total			**48.0**	**28.3**	**7.5**	**2.8**	**9.4**	**332.3**	

Exchange rate (Oct.'88): $1.00 = Cedis 229.

TABLE 11 GHANA HIGHWAY AUTHORITY, PLANNING DIVISION, TRUNK ROAD MAINTENANCE BY CONTRACT—SUMMARY OF ICB PHASE 1, SEPTEMBER 15, 1989

NO.	ROAD NAME/(Section)	Region	Surf Type	Length (km)	Proposed Works (km) Regrav	Reseal	Resurf	Misc	Expected Cost ($) FC(m$)	LC(m$)	Total($)
	LOT NUMBER 1										
1	Adomi-Asikuma	Eastern	P	26.3		22.8	3.5				
2	Adomi-Asikuma	Eastern	P	9.5			9.5				
3	Ho-Denu	Volta	P	102.5		31.4	71.1				
4	Ho-Anyinawase	Volta	P	19.4		11.8	7.6				
5	Asikuma-Anyinawase	Volta	P	24.0		19.6	4.4				
	Subtotal (Lot 1)			**181.7**	**0.0**	**85.6**	**96.1**	**0.0**	**3.3**	**980**	**1,870**
	LOT NUMBER 2										
1	Koforidua-Asesewa	Eastern	P	45.9		4.3	41.6				
2	Nkuraken-Adukrom-Tron	Eastern	P	36.5		20.7	15.8				
3	Tron-Somanya-Kpong	Eastern	P	16.1		1.8	14.3				
4	Nanfe-Akropong-Adukron	Eastern	P	10.9		8.9	2.0				
5	Koforidua-Adawso-Nanfe	Eastern	P	32.1		11.0	21.1				
6	Adomi-Kpong	Eastern	P	10.0		1.6	8.4				
	Subtotal (Lot 2)			**151.5**	**0.0**	**48.3**	**103.2**	**0.0**	**3.2**	**996**	**1,853**

resurfacing were completed. Also, about 98 km of major rehabilitation works was completed. In 1989, 1 143 km of regraveling, 13 km of resealing, 115 km of resurfacing, and 64 km of major rehabilitation works were completed. The effect of these works on the road condition is presented in Table 13, which fits the 10-year Network Stabilization Plan (Table 7) a lot more than the 7-year plan.

Although the condition of paved roads remained virtually static, there were vast improvements in the condition of gravel roads. It is observed that regraveling is being executed mainly by LCB contractors, whereas resealing and resurfacing are to be executed predominantly by ICB contractors. The current position is therefore explained by the fact that ICB contracts have delayed considerably.

CONCLUSION

After publication of the World Bank's policy study on *Road Deterioration in Developing Countries* (*1*), Ghana is perhaps the first country that has addressed itself to the need for developing a practical program for clearing the backlog of road maintenance and preventing further loss of the valuable investments made in its infrastructure.

The program is developed in such a way that the backlog of maintenance will be cleared on the gravel roads in 5 years and on the paved network in 7 years. The portion of the network in good condition will improve from about 30 to 70 percent, whereas at the same time the portion in poor condition will reduce from 40 to about 10 percent in 7 years. The maintenance expenditure required thereafter will stabilize to within manageable levels annually to maintain the network at the desirable condition level. The program is in its second year of implementation and is already proving to be highly successful.

The key elements of the program that have made it successful so far may be identified as follows:

1. The program is spread over a realistic time period and takes into account the continuing deterioration of the roads during this period.
2. Although the funds required for implementing the program are greater than past allocations under government budget, they are not unrealistically high with external financing. In addition, the plan is accompanied by a doubling of local funds and a commitment by government to leave the project intact in its current and future public investment programs.

TABLE 12 GHANA HIGHWAY AUTHORITY, 1989 THROUGH 1991 PERIODIC MAINTENANCE EXPENDITURE PLAN (million cedis)

ITEM	TOTAL	1989					1990					1991				
		1st Q	2nd Q	3rd Q	4th Q	Total	1st Q	2nd Q	3rd Q	4th Q	Total	1st Q	2nd Q	3rd Q	4th Q	Total
Works already Awarded:																
On-going works ('88)	48	12	36			48										
LCB Phase 1	2,914	600	758	934	622	2,914										
LCB Phase 2	2,215	36	56	106	531	729	531	531	106	318	1,486					
Planned Awards:																
LCB Phase 3 (a)	1,864				460	460	460	460	84	400	1,404					
LCB Phase 3 (b)	4,900						700	700	350	700	2,450	700	700	350	700	2,450
LCB Phase 4	4,250											1,275	1,275	425	1,275	4,250
	200	20	20	40	120	200										
Other Works (c):																
1990	3,684						1,105	1,105	369	1,105	3,684					
1991	3,800											1,140	1,140	380	1,140	3,800
ICB Phase 1 (d)																
For. Comp.	6,069						1,524	909	615	909	3,957	909	909	294		2,112
Loc. Comp.	4,460						1,115	669	446	669	2,899	669	669	223		1,561
GRAND TOTAL (Cm)	34,404	668	870	1,080	1,733	4,351	5,435	4,374	1,970	4,101	15,880	4,693	4,693	1,672	3,115	14,173
For. Comp. ($m)	22.7	0	0	0	0	0	5.7	3.4	2.3	3.4	14.8	3.4	3.4	1.1	0.0	7.9
Loc. Comp. (Cm)	28,335	668	870	1,080	1,733	4,351	3,911	3,465	1,355	3,192	11,923	3,784	3,784	1,378	3,115	12,061
Financing Plan:																
Balance from 1988	734	261	449	24		734										
World Bank/IDA (e):																
For. Comp. ($m)	22.7						5.7	3.4	2.3	3.4	14.8	3.4	3.4	1.1	0.0	7.9
Loc. Comp. (Cm)	1,566	157	161	256	933	1,507	59				59.0					
Road Fund	7,610	250	260	800	800	2,110	625	625	625	625	2,500	750	750	750	750	3,000
Funding Gap (Budget Topping Up)	18,484	0	0	0	0	0	3,227	2,840	730	2,567	9,423	3,034	3,034	628	2,365	9,061

Notes: (a) Amount already tendered: CM 1715 and ready for tendering : CM 149
(b) To be tendered November 1989 and June 1990
(c) Consists of Improvement; drainage; town roads, bridge maintenance works and MMU
(d) ICB Lot 9 (Tamale - Makango Rd. and Tamale town roads) not included
(e) Balance to IDA 1858: $24.5m + credit 1601: $4.0m

TABLE 13 EFFECT OF 1988 AND 1989 WORKS ON ROAD CONDITION

YEAR	ITEM	PAVED 5782			MAJOR REHAB	GRAVEL 8352		
		GOOD	FAIR	POOR		GOOD	FAIR	POOR
1988	Starting Position	1,593	1,236	2,953		2,672	2,989	2,691
	Percent Distribution	28	21	51		32	36	32
1989	Without 1988 M & R	1,434	1,221	3,127		2,290	2,944	3,118
1988	Actual M & R	273	-93	-82	-98	660	-110	-550
1989	With 1988 M & R	1,707	1,128	2,947		2,950	2,834	2,568
	Percent Distribution	30	20	50		35	34	31
1990	Without 1989 M & R	1,536	1,138	3,108		2,529	2,850	2,973
1989	Actual M & R	193	-13	-115	-64	1,143	-296	-847
1990	With 1989 M & R	1,728	1,125	2,929		3,672	2,554	2,126
1985	Percent Distribution	30	19	51		44	31	25

3. The World Bank–financed project includes a technical assistance component that helps fill the gaps in skilled manpower required in the Ghana Highway Authority for the implementation of the project.
4. Local consultants are being used and the local construction industry is being developed and strengthened under the project.
5. About 90 percent of periodic maintenance is by contract and awarded on the basis of competitive bidding.
6. The project is being closely monitored and supervised by the Ghana Highway Authority with the assistance of local and foreign consultants.

Some events at the implementation stage contributing to delay include

1. Lack of cofinancing, and
2. Long contract award process for ICB contracts.

As a result of these, the 1988/1989 project has shifted to 1989 through 1991 and the network is likely to stabilize in 10 years instead of 7 years as originally planned.

The condition of gravel roads has improved according to plan because these have been executed on a timely basis by local contractors. Because of the delay in executing ICB contracts (mainly on paved roads), there has been no improvement in the road condition of paved roads.

ACKNOWLEDGMENT

The work described was possible through the assistance of several people. However, special mention is made of Anil S. Bhandari, who was attached to the Ghana Highway Authority for about 2 years and spearheaded the study with staff at our Planning and Road Maintenance Divisions headed by M. G. Anafi and Joe Lamptey, respectively.

For the preparation of the manuscript, appreciation is due to John Anthony and Ama Botsewa Abbey Sam.

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Highway Engineering and Technology Transfer

Role of Technology Transfer Centers in Developing Safety Programs for Low-Volume Roads

EUGENE M. WILSON AND MARTIN E. LIPINSKI

In spite of the concern for improving safety on the nation's streets and highways, there continue to be unmet safety needs. The increase in tort liability claims has highlighted the need to develop programs and procedures to reduce accidents and minimize the exposure of governmental agencies. Local agencies are responsible for over 60 percent of the roadway mileage in the United States. In excess of 98 percent of this mileage can be classified as low-volume roads as volumes are below 2,000 vehicles per day. These agencies must establish ongoing safety programs by integrating safety considerations in all aspects of development and operation of low-volume roads. The resources of the technology transfer (T^2) center programs in the various states can assist local governments in developing comprehensive safety programs. Various safety-related activities of the T^2 centers, including workshops, newsletters, videotape lending libraries, computer software development, and other related programs, are described. Twelve guidelines are included for developing a systematic safety program. These guidelines can be used by local agencies to achieve highway safety objectives.

Providing a safe driving environment on low-volume roads requires that safety issues be made a part of all decisions that affect the design, construction, operation, and maintenance of roadway facilities. The increase in tort liability claims has highlighted the need to develop programs and procedures to reduce accidents and minimize the exposure of local jurisdictions to large financial losses through court action. The resources of the technology transfer (T^2) programs in the various states can assist local agencies in establishing an ongoing safety program by using a systematic approach to integrate safety considerations in all stages of development and continued operation of low-volume roads. These T^2 programs offer many individual courses and activities that address safety needs and demonstrate how resources can be integrated into an agency's comprehensive safety program and can contribute to achieving safety objectives. Safety has always been one of the principal objectives of highway programs. In spite of this concern, there are unmet safety needs and safety deficiencies must be addressed. In many instances, important considerations of roadway consistency or driver expectation have been violated. Additional problems have resulted because safety issues have been ignored in completing construction or maintenance projects. In other instances, the safety implications of locating traffic control devices have not been reviewed adequately. The key to eliminating these problems is establishing a safety philosophy that is applied to all aspects of an agency's activities.

Local agencies need to advocate safety at all levels by implementing proven programs and by making a conscious effort in all phases of low-volume road activities. Improving the safety of the driving environment and reducing the costs associated with accidents are continuing concerns. Selling safety is then the primary task of the local road manager. Unlike counterparts in highway departments and state departments of transportation, where functional divisions exist, the local road manager has the direct safety responsibility for all roadway tasks. Safety must become an attitude of all low-volume road users and local road agencies.

In the United States, almost 69 percent of the total rural roadway mileage is the responsibility of local agencies (see Table 1) (*1*,*2*). Over 98 percent of this mileage carries less than 2,000 vehicles per day. Almost 60 percent of the riding surface in rural areas consists of unpaved roads. The fatal accident rate on local, non-federal-aid rural roads in 1987 was 4.30 fatalities per 100 million vehicle-miles, twice the rate of 2.15 fatalities per 100 million vehicle-miles for the entire U.S. highway system (*3*).

Compounding the problem at many local agencies is the lack of trained professional engineers. In Wyoming, for example, 40 percent of the counties do not have a county engineer (*4*). These county agencies must rely on part-time consultants to produce site-specific engineering designs. This approach limits the county's ability to deal with safety improvements in a systematic manner. Most local agencies, however, are primarily concerned with maintenance and operational issues. An improved ability to recognize safety deficiencies is needed.

T^2 PROGRAMS

In 1982, the federal government recognized the unique needs of the local road and street system in the provisions of the Surface Transportation Assistance Act. The Rural Technical Assistance Program (RTAP) was created and funded at a level of $5 million per year. There were three objectives:

1. To enhance or establish programs for transferring technology to local agencies;
2. To improve communication on technical transportation issues between federal, state, and local agencies and universities; and

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TABLE 1 LOCAL ROAD AND STREET SYSTEM (*1,2*)

Nonfederal-Aid Mileage

System	Mileage	Percent of Total U.S. System
Rural	2,473,000	64%
Urban	548,000	14%
Local Jurisdiction Mileage	2,673,000	69%

Local Road System Volume

(ADT)	Mileage (000)	Cumulative Percentage
<50	1,129	41%
>50 to <200	831	73%
>200 to <500	343	86%
>500 to <2000	343	98%
>2000	41	100%

Surface Type

Type	Mileage(000)	Percentage
Paved	1,558	58%
Unpaved	1,114	42%

3. To encourage implementation of the latest transportation technologies.

Statewide efforts were begun in 1983 as part of RTAP with the establishment of T² Centers (*5*). There are now 46 T² Centers. These centers, each with its own identity, provide six common services;

1. Develop and maintain a statewide mailing list,
2. Publish a quarterly newsletter,
3. Supply technology materials,
4. Provide an information service,
5. Conduct or arrange at least 10 training workshops or seminars per year, and
6. Perform periodic evaluations of their operations (*5*).

A primary characteristic of the T² Center's activities is that it is consumer oriented, delivering programs directly to the users. Training through T² programs has enhanced local agency understanding and concern about traffic safety problems and their solutions. Early T² needs assessments indicated that many of these jurisdictions did not perceive traffic safety as a problem. RTAP has helped local agencies identify potentially hazardous locations, assess where nonstandard traffic control devices or practices are being used, and provide suggestions for low-cost alternatives to improve overall safety of the local road system.

However, much more can be accomplished. For example, many local agencies do little, if any, analysis of traffic accidents (*6*). Given their resource limitations, accident analysis does not receive a high priority when compared with the other activities that must be addressed in day-to-day operations. Liability is a real concern, but a systematic approach to traffic safety is often lacking.

T² SAFETY ACTIVITIES

The contributions of the technology transfer programs in the area of safety include a wide spectrum of activities. Several examples of specific programs are highlighted in the following paragraphs. These examples point out a growing emphasis on the topic of safety and are representative of the programs offered by the T² centers from coast to coast. Although similar examples could be obtained from most T² centers, it is important to note that each center acts as an individual entity and that the focus of a particular center may not emphasize local road safety to the same degree as one in another state.

Video Tape Lending Libraries

In Maine, as in most T² Centers, a comprehensive video tape library is maintained (*7–10*). These video tapes can be checked out for free, generally for a 2-week period, by local officials and used for in-house training. Among the safety titles in stock in Maine are "Night Time Traffic Control," "Traffic Control for Short-Term Work Zones," "Flagger Training," "Local Bridge Program," and "Railroad-Highway Grade Crossings." Many local agencies schedule regular weekly training sessions throughout the winter and reserve tapes for each week during the season when they cannot work in the field.

Newsletters

The T² quarterly newsletters are frequently used to present articles emphasizing the need for improving safety features or to identify the liability risks resulting from roadway system failures. Examples detailing the consequences of failing to post and maintain load limit signs on county bridges, improper signing and not placing guardrail at warranted locations are typical topics which appear in these newsletters (*11–15*).

Newsletter articles also feature local success stories. One example, demonstrating how public participation can improve road safety, appeared in the Montana newsletter. In Great Falls, Montana, a city with 400 miles of city streets, the city promised to fix all potholes within 72 hr of notification. The residents were asked to call PARTNER and the city crews devoted their time fixing instead of locating potholes (*9*). In Kentucky, the results of a 5-year accident study by counties was presented in the *Research Link* (*16*). This was the first effort by T² Centers to provide comparative in-state data to local jurisdictions.

Most newsletters also contain articles that are instructive in nature. In Tennessee, an article highlighted bridge safety and the use of state accident reports (*17*). In 1984, *Tech Transfer* in California reported traffic sign improvement produced the greatest safety benefit-cost ratio for local expenditures (*18*). In the *Vermont Local Road News*, an article focused on the need to examine locations with the highest accident potential—intersections, horizontal curves, and bridges. Techniques such as standardizing signing, select tree removal, and low-cost sight distance improvements were emphasized. The importance of performing roadway evaluations after accidents has also been stressed in many articles (*8,13*). Many northern states have emphasized the need for a snow and ice control policy (*19*). The common elements stressed in all articles of this type are the importance of notifying the public and establishing an accident record keeping system. Tennessee's *Roadtalk* article, "Safety Pays Off in Dollars, Time and Lives," discussed how Lawrenceburg, after 3 years into a safety program, had reduced damage claims by 50 percent. *Roadtalk* also featured articles that showed the benefits from providing safety clothing to maintenance employees in Washington County and another informing local jurisdictions of available funding sources (*11*). A specific program described was one of the grants from The Tennessee Department of Transportation for signing and traffic engineering (*15*).

Teleconferences

Nationwide teleconferences have been held providing training on topics of "Tort Liability," "Planning a Safety Program," "Work Zone Traffic Control," and "Commercial Driver's Licensing." These emanated from locations around the country and were available at satellite locations in all states.

Computer Software Development and Distribution

Even at the local level, computers have become commonplace and are used to assist in the performance of administrative tasks. Many T² Centers have either developed computer programs or have obtained software for public distribution that could be used to assist in safety evaluations. New Hampshire and North Carolina have developed programs tailored to local agency activities (*20*). Road Surface Management Systems (RSMSs) are examples of the type of programs that have been produced. These RSMS systems are used to evaluate road surface conditions and aid in specifying network maintenance practices and costs.

Workshops

Workshops are one of the primary means used by T² Centers to distribute information on safety practices. In a 1988 survey of 43 T² Centers (E. M. Wilson, unpublished), it was determined that the average number of workshops held during the previous year was 21 workshops per center with over 50,000 man-days of training provided nationwide.

The Wyoming T² Center has been active in promoting workshop activity. Since 1986, it has held 40 one-day workshops in 14 cities devoted specifically to traffic safety improvement topics. Topics have included the following: "Safety Features for Local Roads and Streets," "Tort Liability," "Traffic Engineering," "Safety Studies," "Safety Improvement Programs," "Work Zone Traffic Control," and "Details on Work Zone Traffic Control." As a result of workshop training, several counties have begun safety programs and sign inventory systems (*21*).

The key feature of these T² workshops is that they provide low-cost training for local agency personnel at convenient locations. Before T² training activities, only a limited number of states had local programs and most workshop opportunities required airline travel, overnight stays, and significant registration fees. This limited participation and restricted attendance to only those whose agencies had sizable budgets.

Local agencies have been strong supporters of the workshop programs and have recognized the benefits of having their people attend the sessions. The following quote from Harry Bisco, a local official, which appeared in the September 1987 *Alabama Transportation Newsletter*, illustrates the perceptions of these workshop opportunities (*9*):

> Every association or profession offers training courses that are taught by the leaders in their field, most of whom are volunteering their services in order to keep the cost down. Most fees are used for lodging, food and transportation, but even those costs have risen with inflation. Many municipalities feeling the financial crunch are reluctant to fund training; however,

the dividends returned to them in savings as a result of such training are many, many times the cost. Using the cost of training as an excuse not to send an employee to a school is doing a disservice to the community. I have never attended a conference where I have not saved my municipality several times the cost to attend. Last year, I was engaged as an Expert Witness in three lawsuits against three different municipalities. My job was to testify as to what were the standard procedures followed by a majority of municipalities when plowing snow. I was instructed to find how many courses on snow removal were offered in the last five years within 100 miles of the municipal employees being sued, and what the costs were of such courses. There were courses conducted by the Department of Community Affairs of that state each of the five years. One was 15 miles from the community being sued. Due to the fact that these employees did not follow accepted practice, and had not attended any courses at any time, they were found negligent. The judge admonished the employees, and said that it was no excuse that the community could not afford to send them. They were told that they should have paid their own way, as they have as much responsibility to know the latest techniques as their elected officials. In all three cases the question of punitive damages was brought up, and the responsibility was placed on the employee for not obtaining the necessary training to do the operation efficiently with the latest techniques. Training makes leaders out of men; and leaders are more likely to get promoted than followers. The great Henry Ford once said, the best investment a man can make is in himself! The value of training is PRICELESS!

Other Training Materials

T² Centers have also developed other training materials and compiled reference documents for use by local agencies. These materials are characterized by being focused on the specific needs within a state. Inspector's Job Guides (*22*), providing practical information on key activities, highway maintenance tables, and adaptation of PASER (Pavement Surface Evaluation and Rating) (*23*) to gravel roads are examples of local T² activities that have national application. As the preceding paragraphs have indicated, T² Centers are a valuable resource for local agencies in the development of a systematic safety program. Table 2 presents a summary of the services provided by the 44 T² Centers that were in existence in 1987. Comprehensive listings of the programs offered on a nationwide basis are available from the National Highway Institute and American Public Works Association's publication, "State and Local Highway Training and Technology Resources" (*24*).

TABLE 2 1987 SERVICES PROVIDED BY T² CENTERS

Service	# Centers
Video Library	44
800 Phone-In-State	21
Van Program	11
Electronic Bulletin Board	7
Software Library	17
Retired Engineers' Roster	10
Free Publications	44
Loan Publications	28
Training Workshops	44

SYSTEMATIC SAFETY PROGRAM GUIDELINES

The examples cited provide ideas for local agencies to incorporate in a systematic safety program. The following guidelines for developing such programs on a local level have been formulated based on the experiences of the T² Centers. Elaboration has been made for items not previously discussed.

1. Develop a safety attitude: Consider safety from all points of view and for all tasks. This should begin with the basics—wearing a hard hat and safety vest (supervisors, too)—and should permeate all aspects of an agency's activities, beginning with planning and ending with review and evaluation of completed projects.
2. Plan for safety: This begins with the budgeting process and should include, at a minimum, funds for a roadway inventory—bridges, signing, surface condition, physical characteristics, and salaries to hire local engineering college students for summer data collection and monies budget for employee training.
3. Create a public-private partnership: Seek the input of private citizens and local industry in the identification of safety problem areas. Two key elements are a formalized citizens advisory committee and an active program to keep the communication lines open.
4. Use the resources of the state's technology transfer center: The preceding sections have described the types of resources available through the T² Centers.
5. Classify the roadway system: Work towards completion of an inventory based on classification. Use traffic volume and user/trip characteristics—start by using existing knowledge of high volume and low-volume routes. Identify routes with high truck use and routes that have a high potential for nonlocal traffic. Expand this effort when additional funds become available.
6. Involve maintenance personnel initially in development of safety programs: Stress the value of their input in the program, consider establishing an employee incentive program, and use their knowledge in identification of safety problems.
7. Develop a list of roadway safety deficiencies: The fear of identifying existing deficiencies (liability) should not be considered a problem. The program will develop a systematic attack to eliminate these problems.
8. Develop reasonable and desired safety improvement standards based on classification and need: Consider using approaches such as Kansas' *Handbook of Traffic Control Practices for Low-Volume Roads* as a reasonable (Level 1 safety improvement) for low-volume roads (*25*). Comply with the *Manual on Uniform Traffic Control Devices* (MUTCD). Develop a program to upgrade all facilities to MUTCD standards (*26*).
9. Document everything: Documentation should include the training in the safety improvement program, the priorities

established, the improvements as they are made, all maintenance activities and inspections, and evaluations.

10. Develop an inspection program: Systematically view the roadway systems and traffic control devices. Inspections must be made at night and with opposing roadway traffic to determine the effectiveness of traffic control devices.

11. Use traffic accidents/citizen complaints: View these data as identifying potential system deficiencies that may have been overlooked. Respond to citizen input quickly. Communicate back in a positive manner. At a minimum, use a pin or spot map of all accidents. Conduct site investigations for all serious accidents.

12. View the safety program correctly: Work towards completion; it will never develop if the first step is not taken. Begin with training. This 12-step program will lead local road agencies towards providing a safer transportation system. It is the foundation for integrating safety considerations systematically in all aspects of a jurisdiction's work.

SUMMARY

Local agencies with responsibility for operations and maintenance of low-volume roads must develop comprehensive safety programs to reduce the risk of tort liability claims and to reduce accidents. The T² centers in the various states have ongoing programs that can assist these local governments in achieving their safety objectives. The T² Center programs are consumer oriented and are responsive to the particular needs at the local level. The products of the T² Centers are varied and include workshops, newsletters, lending libraries, and other related activities. They provide low-cost and convenient support to local agencies and provide the means by which safety objectives can be met.

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Local Road and Street Safety Improvement Program

JOHN M. MASON, JR., KAY FITZPATRICK, AND WALTER P. KILARESKI

A project to encourage the establishment of a viable safety program at the local agency level, rather than to identify and correct site-specific problems, is described. Specific project objectives were (a) to develop and present a 1½-day short course that emphasized the benefits of a local road and street safety improvement program; and (b) to conduct on-site visits to assist local agencies in developing, implementing, and maintaining a local road and street safety program. The developmental tasks and specific contents of the short course are summarized and the accomplishments of the on-site visits are described. The contents of the overall program address low-volume, local road and street safety programming; specific safety improvement considerations; elements of a comprehensive local safety program; identification and implementation of feasible countermeasures; economic evaluation of alternative actions; and risk management techniques to reduce roadway tort liability.

A project that encouraged the development of a local road and street safety improvement program (*1*) for municipalities in Pennsylvania is described. The main focus of the project was to encourage the establishment of a viable safety process at the local agency level, rather than the sole identification and correction of site-specific problems. Specific project objectives were as follows:

- Develop and present a 1½-day short course that emphasized the benefits of a local road and streets safety improvement program; and
- Conduct on-site municipal visits to assist local agencies in developing, implementing, and maintaining a local road and street safety program.

The developmental tasks of the short course materials (notebooks and visual slide production), conduct of the 1½-day short course, and accomplishments of the on-site visits are summarized. The findings from the on-site interviews are described, and the evaluation responses of the short course and site visits are synthesized.

PROJECT ACTIVITIES

Background

In order to fully address the broad range of safety issues on low-volume, local roadways, the Pennsylvania Transportation Institute (PTI) was selected to supplement the efforts of the Pennsylvania Department of Transportation's (PennDOT's) Center for Highway Safety. Although PennDOT continues to assist municipalities with information and guidance related to local road safety, PennDOT's safety design liaison engineers were not able to devote sufficient time to identifying and solving local highway safety problems. In order to supplement PennDOT's efforts, the project was divided into two phases. The first phase consisted of developing, producing, and conducting a 1½-day short course for representatives from local governments. The second phase involved on-site visits to interested local agencies.

The overall program also emphasized the benefits of practicing a strong risk management program to reduce roadway tort liability implications. The risk management considerations included the following:

- An explanation of municipal roadway liability per state legislation;
- An encouragement of public official support to promote risk management techniques;
- The need and benefits of data collection, data maintenance, and record keeping;
- An exposure to the legal procedures involved in filing, preparing, and defending a roadway-related lawsuit.

Municipality Selection for Short Course

A joint meeting between representatives of PennDOT's Center for Highway Safety, Municipal Services, and Office of Research and Special Studies; Pennsylvania Transportation Institute; and the Pennsylvania Rural Technical Assistance Program (RTAP) was held early in the first phase to clarify project objectives and to select the participating municipalities. Before the meeting, PennDOT identified 150 rural and urban municipalities with high accident frequencies. Municipalities were selected during the meeting on the basis of the following criteria: accident frequency, population (15,000 to 25,000), a perceived need for this type of program, past interest in programs of this nature, an adjacent growing suburban area, a clearly defined organizational structure, probability of success for implementation, and geographical representation of the state. A goal of at least 30 interested agencies was established initially.

An invitation letter was mailed by PennDOT to the selected municipalities. The letter indicated that PennDOT would provide all accommodations and materials. Approximately 2 weeks after the letters were mailed, PTI began contacting the munic-

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ipalities by phone to determine which municipalities were interested in sending a representative to the course and, for each of these, the name of the representative. Approximately 1 month before the course, 25 municipalities had committed to attendance.

Short Course Development

The "Local Road and Street Safety Improvement Program" short course was directed to the municipal level of road and street safety. The course was designed to familiarize local municipalities with road and street safety improvement programs. Topics of discussion included an overview of a safety improvement program, a municipality's safety responsibilities, the characteristics of a safe roadside, procedures for conducting engineering studies, identification of feasible improvement countermeasures, financing and implementation of identified improvements, risk management, and tort liability.

The goals of the course were to present the benefits of a road and street safety improvement program, help the municipality develop a program, establish priorities, and make efficient use of available funds. Related references and existing materials were reviewed for application to the short course notebook and lectures development. The materials included technical publications and reports, existing photographic slides, related accident information, and risk management reports and analyses. Related short course materials developed by other agencies or institutions were also selectively reviewed. The majority of the information contained in the course notebook was from the following four federal publications:

- *Local Highway Safety Studies—User's Guide*, U.S. Department of Transportation, FHWA, National Highway Institute, Washington, D.C, July 1986.
- *Local Highway Safety Improvement Program—User's Guide*, U.S. Department of Transportation, FHWA, National Highway Institute, Washington, D.C., July 1986.
- *Functional Requirements of Highway Safety Features*, U.S. Department of Transportation, FHWA, National Highway Institute, Washington, D.C., June 1984.
- *Guide to Safety Features for Local Roads and Streets*, U.S. Department of Transportation, FHWA, National Highway Institute, Washington, D.C. undated.

Two notebooks were developed: a Participant's Notebook and an Instructor's Notebook. The text of the Participant's Notebook was typed to cover only 75 percent of the horizontal page. The remaining 1½ in. of space on the right side of each page provided an area for the participants to make notes during the lectures. The text of the instructor's notebook was identical to that of the Participant's Notebook. The extra 1½-in. space along the right margin served two purposes: (a) the slide numbers that corresponded to the text were located in the extra space; and (b) the Instructor's Notebook maintained the same page numbers as the Participant's Notebook. These features enabled the instructors to readily refer the participants to a particular page in the notebook. Text material relating to a specific slide was highlighted in the Instructor's Notebook.

Short Course Presentation

The 1½-day short course was presented on February 14 and 15, 1989. The course materials, lodging for the night of February 14, and meals were provided at no cost to the participants. Twenty-three municipalities sent representatives to the course. Thirty-two individuals attended the short course: 25 municipal representatives, 6 PennDOT personnel, and 1 FHWA representative.

At the conclusion of the short course, evaluation and interest forms were distributed to the participants. A discussion on the evaluation of the course is presented later. The Site Visit Interest Form asked the municipalities to indicate interest in a site visit. Nineteen municipalities indicated potential interest in the site visits.

Site Visit Arrangements

The objective of the site visit was to assist each municipality with the development and implementation of its own local highway safety program. The site visits also made it possible for more local personnel to attend; these personnel could then apply the program content to their respective work areas. As an additional advantage of site visits, questions regarding the storage and use of accident and highway data would be more effectively answered when access to the data is available.

A few months after the short course, most of the municipalities that had indicated a potential interest in a site visit were contacted. Arrangements were made with nine municipalities. During the initial municipal contacts, information was sought on any unique problems identified by individual agencies. However, it was emphasized that the goal of the project was to encourage and assist in the development of a local highway safety program, and that isolated or individual corrective studies would not be conducted. Rather, general guidance in establishing procedures for identifying appropriate countermeasures at high-accident locations would be provided.

Two areas were typically discussed as potential items to be included on an agenda: presentation on selected chapters from the notebook and a review of the record-keeping system. The review of the municipality's record-keeping systems would include an examination of current procedures through discussions generated by a questionnaire, a discussion of potential changes and improvements, and a demonstration of available software that could assist in the management and storage of data records.

Site Visit Administration

Two basic agendas were developed for the site visits: a 1-day agenda and a 2-day agenda. Each visit was conducted with one of these agendas or with minor variations from these agendas. The 1-day agenda was divided into two sessions. The first session, which typically lasted 3 to 4 hr, consisted of a review of the record-keeping systems for the municipality, discussion of potential changes and improvements in the record-keeping procedures, and demonstration of available software that could aid the municipality in its record keeping. The second portion of the visits was the lecture titled "Risk Management to Reduce Roadway Tort Liability."

The 2-day visits expanded the 1-day agenda to include lectures on other chapters from the Participant's Notebook. Chapter 2, "Road and Street Safety Improvements," and Chapter 3, "Local Road and Street Safety Improvement Programs," were typically the requested additional chapters.

An evaluation questionnaire was developed for use during each site visit. This questionnaire directed the record-keeping review. Results from the questionnaire are summarized in the following section.

The microcomputer software demonstration began with an overview of the software. The software presented to the municipalities included dBASE III, Lotus 1–2–3, HISAFE, and HISAM. [dBase and dBase III are registered trademarks of Ashton-Tate Corporation. Lotus and 1–2–3 are registered trademarks of Lotus Development Corporation. HISAFE and HISAM are public domain software for which the Center for Microcomputers in Transportation (McTrans) at the University of Florida serves as the distributor.]

Lotus 1–2–3 is a worksheet program that can be used for calculations (e.g., accident rates). dBASE III is a data manager program that is conducive to the storage and manipulation of data (e.g., sign inventories and roadway survey). Although both programs facilitate data management, dBASE III is better suited to data manipulation; Lotus 1–2–3 is preferable for computations. HISAFE (Highway Safety Evaluation Program) and HISAM (Highway Safety Analysis Monitoring) software and documentation are available from McTrans for $45 and $40, respectively (1989 prices). HISAFE is designed to evaluate the effectiveness of accident countermeasures, whereas HISAM is used to identify high-accident locations, analyze accident characteristics, and produce standard accident analysis reports.

Simple examples were developed for each type of software. A computer and printer were set up during each visit and used to demonstrate the software programs. Related questions were answered during the demonstrations. Sample printouts were also provided to interested municipalities.

Approximately half of the municipalities visited store accident data records on a structured mainframe computer. Most municipalities have limited flexibility to modify the program or merge other potential inventories (e.g., sign or pavement condition) with the accident data bases. Several agencies expressed interest in a program that could serve as a management tool as well as a data storage and retrieval mechanism. For example, agencies are interested in knowing how many signs are older than a specific number of years or when a particular route's pavement markings were last painted. The capability of merging accident data files with other data bases is of particular interest to municipalities. Several municipalities utilize computers, but are only beginning to touch on the capabilities of the computer as a management and record-keeping tool.

QUESTIONNAIRE DEVELOPMENT AND RESULTS

Questionnaire Development

A detailed questionnaire was developed to aid in directing the record-keeping review at each municipality and to gather information about each local agency's data collection methods, traffic safety programs, and liability concerns. Development of the questionnaire was based on the Participant's Notebook; it also expanded on relevant topics and interests. The questionnaire was designed to have the research team learn specific details about each agency's current policies and procedures. It also served as a vehicle to engage the participants in discussions that typically resulted in group interactions. The topics covered in the questionnaire included the following:

- Background information on the municipality,
- Collection and maintenance of data,
- Identification of hazardous locations,
- Performance of engineering studies,
- Establishment of project priorities,
- Consideration of countermeasures,
- Sources of funding, and
- Consideration of liability.

During each site visit, the research team met with a preselected group of individuals representing the municipality. These individuals were chosen by the agency and typically included members responsible for record keeping. Also present were persons representing administrative personnel, the police department, the department of public works, and the highway department. In most cases, 8 to 10 individuals were in attendance and were met as a group, except at one municipality where one-on-one interviews were conducted. The structure of the meeting was flexible and somewhat formal; a question-and-answer or general discussion session was typical. The sessions had the additional benefit of providing an opportunity for separate municipal departments to interact with each other. In many cases, the questions sparked detailed conversation between municipal personnel regarding current policies and possible improvements.

Background Information

The first section of the questionnaire was devoted to general information on each municipality, including size, number of accidents, population, operating budget, etc. Some of the questions were supplemented with information from PennDOT. Table 1 presents the general information on the visited municipalities. The average population of the nine agencies was approximately 24,000. Municipality population ranged from 9,200 to 40,000 residents. Geographic size ranged between 3.5 and 37.0 mi^2, with the average size being 19 mi^2. Total length of municipal roadway averaged 80 mi for the nine agencies. Annual operating budgets ranged from $2 to $14 million.

The municipalities visited had between one and three accidents per day, including both reportable and nonreportable. Most municipalities indicated that the majority of their accidents were intersection related. Several agencies stated that the more severe accidents, those involving fatalities, usually occurred in rural areas and were typically run-off-the-road accidents.

With one exception, all of the municipalities occasionally used the services of consulting traffic engineers. Most agencies

TABLE 1 GENERAL INFORMATION ON MUNICIPALITIES PARTICIPATING IN SITE VISITS

Municipality Miles[a] (mi)	State Highway Miles[a] (mi)	Size (mi)	Population	No. Employees		Operating Budget ($ million)	No. of Reportable Accidents in 1988[b]
				Total	Police Only		
143.46	47.33	28.0	40,000	100	50	7	563
74.35	51.97	37.0	28,000	140	55	10.5	784
81.68	31.76	13.4	24,000	112	36	13	424
62.44	19.51	16.7	21,000	67	34	4	382
45.02	12.06	3.5	20,000	127	29	8	277
95.03	29.34	9.0	35,000	196	80	14	648
120.47	92.76	27.5	20,000	60	25	5	384
30.41	21.46	25.0	9,200	25	10	2	73
73.32	24.87	13.0	23,000	120	35	8	490

[a]Data from PennDOT's Bureau of Accident Analysis (4-3-86 data).

[b]Data from PennDOT's Center for Highway Safety.

collected limited traffic data, such as spot speeds, using in-house resources, but consultants were usually retained to conduct comprehensive traffic studies.

Collection and Maintenance of Data

This section dealt with current data collection and record-keeping procedures. Frequently, separate departments within many of the municipalities maintained different portions of the records. Responses by an individual representing one department stimulated questions from individuals in other departments. Many municipalities began discussing lines of communication and possible measures for improving current record-keeping procedures during the meeting.

Accident Data

At each municipality, the police department was responsible for accident data. Several agencies had a specific traffic safety group within the police department that was responsible for accident investigation.

Accident locations were generally referenced by direction and distance to the nearest intersection. Several agencies used a grid system to record accident locations. The grid number was used on the accident report or added when the information was coded into a computer. One agency located in an urban area used house numbers and street names as a reference.

Four of the nine agencies reported that they enter information from the accident reports into a computer data base. The remaining five agencies keep hard copies on file. Several agencies file reports on both reportable and nonreportable accidents. One agency does not complete an accident report on nonreportable accidents. All municipalities send copies of reportable accident reports to PennDOT. Those agencies not currently maintaining accident records in a computer data base indicated they were in the process of, or close to, using a computer to maintain accident records. The computer data bases are typically structured mainframe programs used by a consortium of agencies within a geographical area. Individual municipalities using these facilities are unable to make significant program modifications or to merge other data inventories with the accident data bases.

Most agencies formally summarize accident data through a monthly or yearly report. In addition, several agencies use pin maps to flag high-accident locations. Municipal representatives indicated that they can informally identify high-accident locations because of their familiarity with the roadway system.

Traffic Data

The municipalities visited had average daily traffic (ADT) counts on at least a limited number of streets. Most municipalities conduct ADT counts on an "as needed" basis, but a few agencies had regularly scheduled ADT updating programs. ADT information was typically stored in hard copy files; one municipality utilized roadway maps.

Road And Street Data

Road and street information is processed on an informal basis at most municipalities. Because personnel are familiar with the system, certain information is not formally documented. The majority of the local agencies reported that they maintain at least limited sign inventories. Several municipalities retain

work orders and purchase records, whereas others have regular inspection programs. A few municipalities maintain a record of roadway marking locations and repaint all markings in their jurisdiction each year.

Most municipalities indicated that bridge inspections were carried out by PennDOT or a county agency. Pavement condition surveys at most municipalities are conducted on an informal basis. Many programs consist of walking pavements and recording work completed. One municipality contracts the services of a consultant to evaluate pavement conditions and recommend treatments. As mentioned earlier, most municipalities are familiar with roadway characteristics through routine maintenance responsibilities, and much of the information is processed informally.

Hazardous Locations Identification

Typically, municipalities have opinions about the locations of hazardous areas because of familiarity with local roads. Municipalities also use accident records (total accident frequency and severity), citizen complaints, and field observations to assist in identifying hazardous locations. One municipality also considers accident rates on the basis of traffic volumes. Four municipalities regularly maintain a pin map. Two municipalities indicated that the police department observes traffic conflicts at high-accident locations or locations believed to be hazardous.

Citizen complaints are usually handled on an informal basis. Most municipalities document a complaint and forward it to the appropriate personnel for action. In most cases, the work performed or action taken is not formally documented.

Performance of Engineering Studies

All municipalities conduct limited or preliminary engineering studies. The agencies visited collect preliminary data using some of the following sources: complaint files, maintenance records, enforcement records, construction plans, and traffic control inventories. Only one municipality occasionally uses collision diagrams in preliminary investigations.

Police departments are typically responsible for carrying out in-house engineering studies. Most municipalities perform traffic volume studies on an "as needed" basis. All municipalities surveyed conduct spot speed studies; some have conducted ball-bank studies. Several agencies indicated they have conducted sight distance investigations at locations with limited sight distance; others have conducted traffic conflict surveys on an informal basis by having members of the police department observe traffic operations.

Establishment of Project Priorities

The municipalities rated budget considerations as a primary factor in the establishment of project priorities. The agencies also took into account departmental recommendations, severity of the pending problem, annual programs, public pressure, and political considerations. All municipalities indicated that informal benefit-cost considerations were based on the judgment of the individuals responsible for the final decisions.

Countermeasure Considerations

A majority of the municipalities visited rely on informal methods to evaluate countermeasures. None of the agencies considers life cycle costs or performs an economic analysis on proposed alternatives. Most decisions appear to be driven by the availability of funds and the perceived severity of the problem.

Funding Sources

Municipalities were questioned about their use of various available funding sources. Part of each municipality's budget consists of funds from the Liquid Fuels Tax and police fines and penalties. If the agency has bridges as part of its network, funds from bridge postings, bridge inspections, and the 1982 Bridge Bill are used. Few agencies use the Railroad Safety Program. Several municipalities are aggressively using public-private partnerships as a source of funds for roadway improvements.

Liability Considerations

The incidence of accidents involving serious injuries or fatalities was the most frequently discussed concern among various liability issues. Proper documentation of an incident to provide courtroom defense was also frequently discussed. Several agencies expressed discontent with insurance companies independently deciding when to settle a case out of court.

Frequencies of traffic-related lawsuits ranged between one and five per year for the different municipalities. Six of the nine agencies currently have at least one traffic-related lawsuit pending against them. Several municipalities have a trained accident reconstructionist on the police staff. These individuals investigate accidents with a high probability for a lawsuit.

PROJECT EVALUATION

Short Course Evaluation

During the short course, two evaluation forms were distributed to the participants. Issues addressed ranged from the perceived usefulness of the various sections presented in the course, to whether course objectives were met. A summary of the results from each evaluation is provided in the following paragraphs.

The front side of Course Evaluation I gathered general information on the participants. The 25 persons attending the short course represented municipal public works departments (12), police departments and traffic safety units (6), township managers (2), township engineers (2), streets and parks department (1), a technical assistant (1), and an assistant road foreman (1).

The municipal duties of the participants varied from road construction and maintenance to administrative responsibilities. Participants were also questioned regarding the number of years of experience in their respective areas. Answers ranged

from 1 year to 29 years of experience. The following illustrates the distribution of years of experience among the participants:

Number of Participants	*Years of Service*
7	0–5
8	6–10
5	11–15
2	16–20
2	19–25
1	Over 25
25	

Of the 23 municipalities attending the short course, 7 stated that they currently have a specific road and street safety improvement program. Nineteen of the municipalities indicated that they were interested in a follow-up on-site visit to initiate or improve existing street safety improvement programs.

Side 2 of the Course Evaluation I form gathered information on the usefulness of the various sections presented in the short course. Participants were requested to answer this question after the presentation of each section's material. On a scale of 1 (not useful) to 5 (very useful), the results ranged from 4.2 to 4.8. The overall average rating for all presentations was 4.4 out of 5.0. The highest usefulness rating was given to Chapter 6, "Risk Management to Reduce Roadway Tort Liability." All nine municipalities participating in the site visits also requested that this chapter be presented to their agency's personnel.

The Course Evaluation II form dealt with the course content, materials, presentations, and intended objectives. The majority of the participants felt that the objectives of the course were met.

A one-page Site Visit Interest Form asking the representatives to indicate interest in an on-site visit was distributed at the conclusion of the presentations. The form also provided an opportunity for participants to make special comments or requests. Overall, the comments were very positive. Even participants who indicated they were doing essentially the right things found the presented materials supported current policies and procedures. Several participants commented on the usefulness of the information contained in the Participant's Notebook and inquired about purchasing additional copies. One participant recommended developing separate sections of the course, one dealing with rural concerns and another addressing urban concerns. One municipality commented: "Request PennDOT to fund additional regional forums. We would support these updated forums even if we would need to pay."

Site Visits Evaluation

Seven of the nine visited municipalities provided evaluation letters. The letters were very positive regarding this program. Each municipality felt that the time spent was well worthwhile and that the information presented was very useful to them as they develop and maintain their safety improvement programs. Frequently, municipalities stated that the information was presented in a way that was easy to understand and follow. Several municipalities indicated that the instructor's involvement with actual tort claims cases helped to emphasize the importance of a roadway safety program at the municipal level.

The visited municipalities recommended continuing the program and establishing additional programs related to traffic safety. A number of activities currently being implemented or under consideration by participating municipalities are the following:

- "The suggestions and ideas relative to the Township's current record keeping system are currently being reviewed for implementation. We are also exploring the possibility of computerizing the data."
- The afternoon session "also provided us with insight in how to utilize complaints, inquiries, or accident data to review potential problems or hazardous areas or conditions."
- ". . . it provided each township employee an awareness of the potential hazardous and perhaps libelous conditions that can exist—and the need for constant vigilance and action by the employee to identify and correct these conditions."
- "We have already cut away some brush obstructing some road signage and are ordering a ball-bank indicator to determine advisory speeds for curves on some of our rural collector roads. We have also ordered some cone reflectors and additional orange and white barricades for work zones as suggested."
- "The program, as presented, was an important first step in controlling liability risks through the education and training of our employees."
- "The most important aspects that were missing (in the municipality) included a comprehensive accident investigation and reporting document, and the need for a complaint response system that incorporates documentation of response and automatic follow-up. These two practices have already been implemented by the Public Works Department."
- ". . . (this program helped) to raise the conscientiousness level of our staff . . ."
- ". . . township is proceeding to acquire a computer road/street traffic identification and maintenance record system and hopes to have it in place by 1990 to use as a management tool for a road maintenance/safety program."

SUMMARY OF FINDINGS

The Local Road and Street Safety Improvement Program short course presented on February 14 and 15, 1989, in State College, Pennsylvania, had 32 participants that represented 23 municipalities, PennDOT, and FHWA. Site visits were held at nine municipalities.

The significant findings and results of the overall project are as follows:

1. The participants rated the usefulness of the short course very highly.
2. The "Risk Management to Reduce Roadway Tort Liability" section received the highest most useful rating of the six sections included in the Participant's Notebook. This section was also requested by the nine municipalities participating in the site visits phase of the project.

3. Noteworthy recommendations from the participants include the following:

- The program should be split into two sections—one geared to *urban* areas and one to rural/developing areas since they have very different needs and concerns.
- This course seemed a bit rushed. Should probably be two full days. Would like to see follow-up courses on specific subjects.

4. The municipalities participating in the site visit program asked a broad range of technical and legal questions. In order to be responsive, PTI was very flexible in the agendas. It was this element of the site visit that was most meaningful to the local agencies. This is true because each local municipality has different needs, issues, and concerns.

5. Almost every local agency expressed a very strong need for more help with microcomputer data base development. Key areas identified include file management/structure; inventory of signs, signals, and markings; pavement condition; and evaluation software. During the site visits, PTI exposed each municipality to HISAM/HISAFE, dBASE III, and Lotus 1–2–3 using some elementary and very limited examples. However, much more needs to be done to effectively assist the local agencies in actual software development using practical and related examples.

FUTURE ACTIVITIES

Based on the reported interest and requested need for further assistance from the municipalities, PennDOT, via its Rural Transportation Assistance Program (RTAP), has hired a full-time safety engineer to continue the site visits to provide the necessary support and guidance. PTI will present additional 1½-day short courses; the RTAP safety engineer will conduct the local site-specific training aspects of the overall program. Additionally, the RTAP safety engineer will develop and maintain an ongoing evaluation of the effectiveness of the program objectives.

ACKNOWLEDGMENTS

This effort was sponsored by PennDOT and funded by the Federal 402 Highway Safety Program. John M. Mason, Jr., served as Principal Investigator. The notebooks were prepared by the Principal Investigator and the following individuals: Walter P. Kilareski, Kay Fitzpatrick, Robert E. Brydia, and Deborah L. Seneca. The on-site visits were generally conducted by John M. Mason, Jr., and Walter P. Kilareski as lead instructors with the assistance of Kay Fitzpatrick and Kevin L. Slack. James P. Tenaglia and William J. Pogash were the contact representatives from PennDOT. Donald J. Jacobs provided guidance on PennDOT's risk management activities. The Pennsylvania Transportation Institute appreciates the cooperation and assistance of all the various members of PennDOT.

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Communicating Change to Local Roads Officials

KATHRYN P. MCDERMOTT AND EDWIN W. HAUSER

Providing technical assistance to local roads managers has not been a priority of traditional federal research programs. However, the national network of technology transfer (T^2) centers, which emerged in the 1980s from the FHWA's Rural Technical Assistance Program, are now at the forefront in providing technology transfer to local roads officials. Accordingly, the T^2 centers' primary responsibilities are to (a) identify the needs of local officials, (b) package the appropriate technology for meeting those needs in such a way that it can be easily understood and implemented at the local level, and (c) follow up on locals' use of the new technology to make sure its implementation has been successful. Although these centers have in the past 10 years made great strides in accomplishing the first two stages of this process, they must continue to work hard to find appropriate means for accomplishing the third. T^2 centers must begin to reassess their role as agents of change and ask themselves, How well are we communicating technological change to local roads officials? The only way to determine if the technology transfer program has been successful is to document how locals have used the technology received through these centers to improve their method of operation. A successful technology transfer effort ensures that new technology is both introduced to local officials and implemented practically to solve a problem or satisfy a need. If a climate conducive to change is not created by the technology transfer agent in his relationship with local officials, then change will not occur. T^2 centers need to develop strategies that will help them not only to learn whether or not local officials are making significant use of the information and materials the centers provide but also to determine the costs associated with implementing technological change for the user's benefit.

Low-volume roads are vital to the fabric of the nation's economy. They make up more than 82 percent of the nation's highways; provide access to Interstates and primary roads; and play an integral role for agricultural, manufacturing, and tourism industries. Yet, rural local roads remain among the nation's most hazardous roads and receive the least attention from the federal government, which traditionally allocates funding toward programs designed to improve the nation's Interstate and urban systems.

In the early 1980s, Congress made the first significant move to recognize the transportation needs of rural areas when it established the Rural Technical Assistance Program (RTAP) under FHWA. From this federal program emerged the technology transfer (T^2) program for local transportation agencies. The national network of T^2 centers subsequently created is now at the forefront in introducing to local roads officials the technological resources available for improving the design, construction, and maintenance of low-volume roads. But as time has shown, closing the gap between available research and the practical needs of local roads officials is hardly an easy job for the nation's T^2 centers.

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COMMUNICATING TECHNOLOGICAL CHANGE

Technology transfer is without question a complicated process whose success is determined by two factors: money and communication. It takes money to conduct the research, harness the professional expertise, and develop the resources necessary to foster technological innovation. With respect to the T^2 centers, the issue of who should provide funding for their program activities is presently under rigorous debate, and will continue to be debated as long as federal funding for these programs is reduced.

However, of equal, if not greater, importance is the second of these two factors, communication, and its role in technology transfer at the local level. With an average budget of over $200,000, and a vast storehouse of technical information at hand, the nation's T^2 centers currently have the means to provide significant training and technical assistance to local governments. Accordingly, T^2 centers must determine to what extent they are willing and able to participate in the entire technology transfer process.

Technology transfer is successful only when technical knowledge has been learned and put to practical use to solve a specific problem (*1*). In other words, technology transfer is a process which must involve change for the common good. After nearly 10 years of operation, most T^2 centers are well skilled at introducing new technology to local users through workshops, demonstrations, newsletters, and other methods. The question now is How successfully are these federally funded T^2 centers communicating technological change to local groups?

NATURE OF LOCAL GOVERNMENT AND THE T^2 CENTER

It has been suggested that the T^2 center and the local groups it serves assume functional roles that at times may pose the greatest impediment to communicating technological change.

After a series of meetings with officials from local municipal and county governments, and after 3 years of operation providing technical assistance, The Center for Local Government Technology at Oklahoma State University (*2*) found that the following characteristics were typical of the nature of local governments overall:

1. Expediency oriented,
2. Autonomy of operation,
3. More services demanded,
4. Tax burden too high,
5. Labor intensive but low pay scales,
6. No tradition of efficiency, and
7. Inadequately trained personnel.

This list suggests that local governments, in short, operate under the "Home Rule Principle." They work in the short term, make their own rules, are primarily responsible for the construction and maintenance of low-volume roads, and therefore neither expect nor particularly appreciate interference from the state or federal government. From a local perspective, federal money inevitably brings federally mandated specifications and regulations that rarely take into account the specific needs of the local agency (*3*).

The T² center, on the other hand, is essentially the agent of change. As an appendage of the federal government, it believes valuable research exists that should be implemented at the local level, not only to standardize construction and maintenance practices, but also to increase safety and cost efficiency in the long term. The T² center sees itself as bearing the fruits of research to local governments that may have neither the money nor the personnel to seek these technological resources on their own.

The above description poses a strictly black and white scenario of the nature of the local agency and the T² center. In truth, local governments are interested in new materials and information that can be used to make their roads safer and more cost-efficient. However, they are not interested in wrestling with federally mandated specifications and requirements that are inappropriate and overly complicated (*3*). The reality of their work environment, as the Oklahoma State University study reveals, will not allow for such red tape. What local governments do want is the flexibility to use and adapt technological resources to meet their own needs. This is where the services that the T² center can provide come into play.

ROLE OF THE T² CENTER AS THE AGENT OF CHANGE

As the agent of change, the T² center acts as middleman between available research at the state and federal levels and the needs of local roads managers. Accordingly, the primary responsibilities of the T² center are to (a) identify the needs of local officials, (b) package the appropriate technology for meeting those needs in such a way that it can be easily understood and implemented, and (c) follow up on locals' use of the new technology to make sure that its implementation is successful (i.e., solves a problem or satisfies an unfulfilled need).

As previously mentioned, the T² centers operating under RTAP are now well familiar with and already making practical use of the techniques available to them for fulfilling these first two responsibilities. Moreover, a generous amount of research exists that has investigated many centers' successes and failures with respect to identifying local needs and packaging technology to meet those needs. NCHRP's recent synthesis on *Technology Transfer in Selected Highway Agencies* (*4*), for example, offers an excellent overview and ranking of the mechanisms currently used by T² centers for transferring technology to local agencies. Of course, with time and growth inevitably comes the temptation to standardize operations for the sake of convenience. There is no question that T² centers must resist the urge to distribute "canned" solutions to local problems on a regular basis. In keeping with its role as an agent of change, the T² center must instead continually challenge itself to seek out new methods and experiment with different strategies for transferring technology to local roads officials.

FOLLOWING UP ON THE IMPLEMENTATION OF NEW TECHNOLOGY

The one responsibility T² centers are not yet fully addressing is following up on locals' use of new technology. Making sure that a new technology is applied effectively once delivered to the user seems virtually an impossible task for the T² centers, which are understaffed yet serving a broad range of users. However, technology transfer, if it is to be fully achieved, does not occur until a new idea or technology has been successfully implemented to the user's benefit. It is essentially the T² center's duty to make sure that the implementation process is sufficiently carried out. As the agent of change, the T² center should engage in activities that ensure that implementation occurs where needed to solve local transportation problems. The following are three suggested tasks that may help the center achieve this goal:

1. Create a climate conducive to technological change,
2. Monitor local efforts to implement new technology, and
3. Promote local success stories concerning the implementation of new technology.

It is the findings obtained from the technology transfer agent's engagement in these three tasks that ultimately determine the success of any technology transfer effort. Before investigating these three tasks further, it may be helpful to first understand how the federal perspective of evaluating technological change differs from the technology transfer center's view. Each view is valuable, and each points to the need for the federal government and the T² center to become partners in solving the problem of program evaluation.

A FEDERAL PERSPECTIVE OF EVALUATING TECHNOLOGICAL CHANGE

From a federal perspective, documenting the dollar savings that result from technological change is a major concern. In a presentation given at the 1990 National RTAP Conference, the executive director of FHWA recognized the difficulty of quantifying the benefits that result through the training provided by the T² centers (*5*). He observed that dollar savings, in particular, obviously exist. Approximately $24 million to date has been saved as a result of the efforts of the nation's T² centers; however, this figure may be just the tip of the iceberg.

At present, the T² centers do not have a standard method of evaluation (MOE) for estimating the dollar savings resulting from its training efforts. Course evaluation forms and annual program evaluation forms provide T² centers with the primary tools they have to evaluate overall program effectiveness. Although these methods keep the centers aware of how receptive locals are to their services, they offer little, if any, quantifiable evidence of dollar savings to the federal government, which provides funding for the centers' activities. In short, the federal government knows that technology transfer programs are saving users' dollars. But they don't know how much. They know that changes are occurring. But they don't know to what extent. According to the worst case scenario, unless MOEs are developed to measure such factors, the federal government runs the risk of spending millions of dollars supporting T² centers that transfer technology that never gets used to its full capacity.

NCHRP's synthesis on *Technology Transfer in Selected Highway Agencies* (*4*) suggests that if quantifiable MOE's are to be developed for the T² centers, they should emerge from research projects initiated by NCHRP or FHWA. These MOEs should quantify the benefits derived from both individual technology transfer and from the technology transfer program themselves. These MOEs may be based on dollar savings or engineering economy analyses but nevertheless should be developed for standard use among the nation's T² centers (*4*). Thus, the federal government would have the information it needs to conduct benefit-cost analyses and justify future funding expenditures for technology transfer program activities.

A T² CENTER PERSPECTIVE OF TECHNOLOGY TRANSFER

However, from the perspective of the T² center, one either believes in the overall value of training or one doesn't. With respect to program evaluation, what the T² centers want or need to know—the bottom line—is not necessarily how many dollars are saved as a result of better training, but whether or not the training they provide is solving the transportation problems that the locals need solved. Developing a formal strategy for evaluating the practical applicability of new technology, like the three-step follow-up strategy previously described, is instrumental in helping T² centers determine whether or not their engagement in the technology transfer process is both comprehensive and successful.

CREATING A CLIMATE CONDUCIVE TO TECHNOLOGICAL CHANGE

Louis B. Stephens, Jr. (of Wilbur Smith Associates), in his article *What Happens When the Consultant Goes Home?* poses four questions that may help the T² center evaluate whether or not the technology transfer effort will yield a significant change resulting in an overall net improvement for the user (*6*). These questions are as follows:

1. Are there recognizable factors within the recipient community that must be present if long-term success of the technology transfer is to be ensured?
2. Are there factors or attitudes that must be present among the donors or providers to ensure long-term impact?
3. Are there recognizable factors that would cause the on-the-scene agent to suspect that a program might fail or have only short-term impact?
4. What type of feedback does the provider require to improve his own experience and how can he ensure receiving such feedback without compromising its value?

These questions imply that the technology transfer agent must continuously develop his or her professional expertise and understand the user's environment and needs to predict the overall success of the technology transfer effort and create a climate conducive to technological change.

North Carolina's T² center, located at The University of North Carolina Institute for Transportation Research and Education, has spent the past year investigating whether or not its efforts have fostered the potential for technological change among local officials. In the past, North Carolina's center, like other centers, has relied chiefly on course evaluation and annual program evaluation forms to obtain feed-

Course Evaluation

First, rate the following:	Circle One (Ten = Highest)									
1. Classroom Instruction	1	2	3	4	5	6	7	8	9	10
2. Course Content	1	2	3	4	5	6	7	8	9	10
3. Training Materials	1	2	3	4	5	6	7	8	9	10
4. Overall Effectiveness	1	2	3	4	5	6	7	8	9	10

In the space below, write your constructive comments for any improvements on any topic area. Please be specific. If you need additional space, please write on the back.

FIGURE 1 Original course evaluation form.

Workshop Title

Workshop Date
ITRE Training Facility, Raleigh, NC

COURSE EVALUATION

Directions: Please rate each course area listed in the box on a scale of 1 to 5 (**5 being the highest**). Then respond to the questions which follow by checking "yes" or "no" and providing additional comments when necessary. Your feedback will enable us to update and enhance this course periodically so that we may better address your needs.

Classroom Instruction	1	2	3	4	5
Course Content	1	2	3	4	5
Training Materials	1	2	3	4	5
Administration (staff performance, registration, check-in, etc.)	1	2	3	4	5
Meeting Facilities (meeting room, acoustics, temperature, etc.)	1	2	3	4	5
Overall Effectiveness	1	2	3	4	5

Classroom Instruction

a) Did the instructor/s/ present information clearly and at an appropriate rate? ____Yes ____ No
b) Did the instructor/s/ effectively communicate the course objective/s/? ____Yes ____ No
c) Did the instructor/s/ respond to participants' questions in a satisfactory manner? ____Yes ____ No
Comments:

Course Content

a) Were the course topics suitable for your background and experience? ____Yes ____ No
b) Which topic/s/ did you find most useful? _____
Least useful? _____

Comments:

Training Materials

Were the training materials a useful supplement to the course instruction? ____Yes ____ No
Comments:

Administration (staff performance, registration, check-in, etc.)

Comments:

Meeting Facilities (meeting room/s/, acoustics, temperature, etc.)

Comments:

Overall Effectiveness

a) Do you feel you can effectively use the information obtained through this course in the workplace? ____Yes ____ No
b) Do you feel this course requires follow-up assistance or additional short courses to help you make practical use of the information presented? ____Yes ____ No
Comments:

Follow-Up

Do you believe that the information received through this course will enable your organization to reduce costs in operations? ____Yes ____ No

Would you be willing to document those savings for us? If so, please give us your name on a separate sheet before you leave.

Thank you for your assistance!

FIGURE 2 New evaluation form.

back from local users; however, because of the nature of the forms, the feedback received was not indicative of whether or not the technology presented through the workshops, newsletters, and other forms of technical assistance could be practically applied. In short, the course evaluation forms explained only how well the local officials were entertained. Once a workshop participant left, he disappeared into a foggy spectrum of "nowhereland," preventing a witnessing of any real use of the technology to which he had been introduced.

In December 1989, North Carolina's T^2 program took steps to uncover, first of all, whether or not the information being offered was of any practical use to the local groups making use of the center's services. In order to accomplish this goal, the traditional course evaluation form used for the past 3 years was revised. Figure 1 shows a copy of the original course evaluation form, which offered locals virtually no opportunity to communicate whether or not they could realistically apply the information they obtained through the center. The new evaluation form (see Figure 2) allows for an overall rating of specific course areas, but also includes questions that direct locals to respond to how they feel the course contributes to their specific needs. Three questions, in particular, were added to address these concerns:

1. Do you feel you can effectively use the information obtained through this course in the workplace?
2. Do you feel this course requires follow-up assistance or additional short courses to help you make practical use of the information presented?
3. Do you believe that the information received through this course will enable your organization to reduce costs in operations?

From January through October 1990, 23 workshops have been held with 780 participants. Of those participants, 623 completed and turned in the newly revised course evaluation forms. For the purpose of this report, their responses to the three questions were calculated and evaluated. The results obtained and conclusions drawn are provided below.

• Question 1: Do you feel you can effectively use the information obtained through this course in the workplace?

Five hundred ninety-eight (96 percent) of the respondents answered "Yes" to this question; 25 (4 percent) answered "No." Clearly, it can be assumed that the local constituents are obtaining information and materials that they feel they can make practical use of. However, according to a few comments, although the information was useable, nothing could be achieved unless upper management was willing to listen and implement new ideas. This concern, noted by several respondents, suggests a need to investigate to what extent local roads managers are aware of and understand the applicability and usefulness of the information their personnel are bringing back with them. It would also be desirable to understand local roads managers' perceptions of training overall. For example, is a workshop merely an opportunity to describe established knowledge and procedures, or is it an opportunity to promote the implementation of change where change is needed; or is it, perhaps, both, and under what situations? Finally, it would be interesting to investigate what percentage of workshop attendees, or users of the center's services, are local roads decision makers. A clearer perception of the rank and titles of workshop participants may indicate to what extent promoting change is likely. The role of the local roads manager in the technology transfer process is certainly an issue worth exploring.

In fact, a clear understanding of audience is essential if a T^2 center is to be able to match available technology with local user needs. North Carolina's T^2 center, like most others, maintains a data base of all workshop participants consisting of their names, titles, organizations, addresses, and population groups. The data obtained from annual evaluations of its local audience can be an invaluable tool for any center that undertakes more extensive analyses of its target audience. For example, for the period January through October 1990, North Carolina's T^2 center, on the basis of information obtained from its data base of workshop participants, found that 52 percent of all workshop attendees came from rural areas (population below 50,000), whereas 48 percent came from urban areas (population of 50,000 and above). Of all rural participants, 61 percent came from communities with populations between 10,000 and 50,000. These figures alone indicate that at present the center's local users are from medium-sized communities and larger. As a result, as it plans for its 1991 training schedule, the center may need to take a serious look at how to reach smaller rural areas that are in need of its services but either unaware that the center exists or unable to travel to the workshops.

The Bay State Roads Program at the University of Massachusetts conducted a more comprehensive analysis of its target audience and presented the results at the 1990 National RTAP Conference (*7*). Their evaluation consisted of data obtained from the following categories:

—Percentage of towns stratified by population,
—Percentage workshop attendance for each population group,
—Workshop participation by location of community,
—Workshop participation by type of personnel,
—Percentage of towns in population group borrowing videotapes,
—Video library use by location of community,
—Roadshow participation by size of community,
—Roadshow participation by location of community,
—Roadshow participation by type of personnel,
—Requests for information by size of community, and
—Requests for information by location of community.

With a well-maintained data base, audience-related information like this may be fairly easy to obtain, though the process is time-consuming. The overall benefit of possessing such information is clearly evident. When precisely who our audience is is known, how to communicate to them in a way that caters to their specific needs, expectations, and beliefs becomes possible. Consequently, relationships are built that foster opportunities for technological change to occur.

• Question 2: Do you feel this course requires follow-up assistance or additional short courses to help you make practical use of the information presented?

Of 623 participants, 418 (67 percent) answered "Yes," follow-up assistance should be provided, while 205 (33 percent) responded "No." According to these figures, at least one-third of our training courses can be considered "stand alone" technology transfer efforts. In other words, participants may come to the training session, obtain the information and materials needed, take them home, and use them, as is, without significant post-training assistance. Although this fact does not necessarily ensure implementation of a new technology, it does imply that the packaging of the technology for local use has been successful. The remaining two-thirds of the workshops, however, do require follow-up assistance of some form to help users make the most effective use of the technological information and materials presented. Of those participants who said "Yes" on the course evaluation forms, the following types of follow-up assistance were suggested for the workshops in question:

—Refresher courses every 12 to 24 months to provide updates on changes in the information and materials presented,
—Additional short courses focusing in more detail on specific topics presented in workshop, and
—Problem-solving or hands-on demonstrations to make information "more realistic and memorable."

This information is tremendously valuable to the center's program. In the past, a single workshop, once developed, was conducted over and over again until attendance levels dropped, indicating a general lack of interest or need. But what the responses imply is the need to begin offering, where appropriate, several levels of courses for such topics as "Geographic Information Systems for Transportation (GIS–T)" and "Site Impact Traffic Evaluation (SITE)" to help local users gradually develop their expertise of a new technology before they begin to investigate ways to advance its implementation in their work community. Furthermore, by offering several levels, or modes (i.e., refresher, short course, hands-on demo, or advanced level), of a specific course topic, the target audience can be pinpointed so that each course is filled with individuals with similar backgrounds and levels of experience. Courses made up of participants with backgrounds too diverse are usually doomed to failure regardless of the value of the technological information presented.

• Question 3: Do you believe that the information received through this course will enable your organization to reduce costs in operations?

Four hundred eighteen (67 percent) participants believed that the information obtained through the workshops would allow them to reduce costs in operations, although none provided any ideas as to how much those savings might be. Where comments were provided, participants believed the savings related more to time and efficiency of operation than money. For example, one participant remarked that the SITE workshop would "save time by improving efficiency of analysis." For the communication workshop titled "Getting Your Message Understood: Communicating with Decision Makers, Citizens, and the Media," one participant remarked that "the cost analysis would be publicity savings (cutting out bad public relations)."

One hundred twenty-seven (20 percent) participants said that they did not believe the information they received would reduce operations costs. In fact, many felt that the course information and materials, if implemented, would require them to spend more money. For example, one participant attending the "Work Zone Safety" workshop said that he would have to spend more money to get the equipment he needed to construct safer work zones, although in the long run, he would probably save on liability costs. The following comment came from a participant of the GIS–T workshop: "My experience has been that GIS does not save costs. What it does is create new products and allow us to provide better service and more information for analysis, but it often requires more time and money." Seventy-nine participants (13 percent) were unsure of the likelihood of cost savings or provided no response.

The responses to this question suggest that T^2 centers are perhaps, indirectly but in many cases, asking local users to spend money (and lots of it) in order to implement new technology. If this is the case, then centers need to become more aware of the initial and long-term costs locals must face if they are to adopt a new technology. Centers must also ask themselves, "How can such information be communicated to locals in a way that would stimulate rather than inhibit the technology transfer process?" There is no question that money is a crucial barrier to technological change for most rural local agencies. But to provide training in new technology without any discussion of how to finance its implementation is like asking a driver to operate a motor vehicle without any fuel. The desire may be strong, but without adequate resources, the task impossible.

By revising its standard course evaluation form, North Carolina's T^2 center now directs locals to comment on the applicability of the new technology presented at its workshops. Thus, the center is able to determine more precisely whether or not it is creating a climate conducive to technological change through its training efforts. On the basis of the responses provided, the center discovered that it is basically introducing locals to the information and materials they need to solve current problems; however, the center also discovered that certain conditions should exist before the new technology may be applied:

—Local roads managers and decision makers must offer their support and express a willingness to adopt technological change.
—The T^2 center must provide follow-up training assistance as needed in the form of refresher courses, short courses, advanced-level courses, and hands-on demonstrations to cater to individual needs and levels of experience.
—Strategies for financing technological change must be explored and presented at workshops to help locals evaluate the resources available to them for implementing a new technology.

With this information in hand, North Carolina's T^2 center is better prepared to strengthen its position as a catalyst of technological change among rural, local communities.

The next step in a T^2 center's follow-up strategy should be to move beyond the level of presenting new technology in a climate conducive to change to a higher level of monitoring its successful implementation. T^2 centers cannot afford to

overlook this important responsibility of making sure that locals implement new technology cost effectively to meet their needs. Too often T^2 centers get stuck in a rut—churning out one training session after another, placing more value in the number of workshops conducted and individuals attending than in the overall rate of adopted technology. The scenario of the workshop participant who takes home a notebook full of good ideas only to let it collect dust on a shelf is all too often the actuality. Centers need to bluntly ask themselves, "How are locals using the information we give them?"

MONITORING LOCAL EFFORTS TO IMPLEMENT NEW TECHNOLOGY

Since it was created 4 years ago, North Carolina's T^2 center has not had the personnel or resources necessary to actively engage in any effort to monitor locals' use of the technology presented through its workshops. Like most newly established centers, its efforts were directed toward creating and maintaining a mailing list and packaging available technology for local use. However, for the 1991 training schedule, the center plans to achieve this new objective by distributing post-training evaluation cards approximately 3 months following each workshop. Of 46 T^2 centers in the country, 11 have already made use of this method of follow-up evaluation, although each may rely on these response cards for different purposes.

North Carolina's T^2 center plans to use these cards to determine how past workshop attendees used the information they received to improve their method of operation. The follow-up response card would include the following five questions with space provided for additional comments:

1. Did the workshop provide you with the information/materials you needed? If no, please explain: YES NO

2. Have you made changes for the better in your method of operation as a result of this workshop? Please explain: (If you answered no, please go on to question 4) YES NO

3. (a) Were the costs associated with implementing these changes extensive? Please explain: YES NO

   (b) What would you estimate to be the costs related to implementing these changes?

4. What prevented you from making use of the information/materials presented through this workshop?

5. How can we improve this workshop for your benefit?

Because the information to be obtained from these cards is so important, the T^2 center can do certain things to improve the response rate. Follow-up response cards should be brief (3 to 5 questions is usually sufficient); they should be mailed directly to the workshop participant (not his department or supervisor) and should, if possible, be produced on a detachable, self-addressed, postage-paid card. The easier the task, the more likely a response.

Aside from follow-up response cards, the center may rely on additional strategies for monitoring locals' use of new technology. Centers can keep the channels of communication open by offering locals an 800 number for calling in questions, problems, success stories, etc., related to their attempts to implement new technology. Locals can be encouraged at the workshops to use this number to keep in touch with the center and let them know how they were able to make use of the information presented at the workshops once they returned to their work environment. Furthermore, the center may conduct its own phone surveys, in addition to written program evaluation forms, to spot-check locals' progress with implementing new technology. These surveys may be done on a quarterly or annual basis.

Whatever strategy, or strategies, a center chooses to use, it should be sure to make contact directly and regularly with the locals who have attended the workshops or requested technical assistance in some other way. In other words, the center should rely consistently on feedback obtained firsthand from its local users—in addition to the suggestions and recommendations put forth by secondary sources like advisory committees, which are isolated bodies that are integral to but not directly involved in the technology transfer process. Only then can the T^2 center witness the true results of its technology transfer efforts.

PROMOTING LOCAL SUCCESS STORIES

T^2 centers have always been encouraged to share their own success stories with other centers concerning innovative training ideas. The Washington, D.C.–based T^2 Clearinghouse and the annual National RTAP Conference foster networking between the centers so that they may improve program efficiency and training methods.

Likewise, the T^2 center should foster networking among the agencies it serves by promoting local success stories in its newsletter. Two advantages result. Locals benefit from the opportunity to interact not only with the T^2 center but with other local agencies that may have transportation needs similar to their own. The T^2 center benefits from the opportunity to document local efforts to implement new technology. A side benefit is that the T^2 center, through its promotion of local success stories, continually nurtures an environment conducive to technological change and strengthens its role as a partner to local agencies in the technology transfer process.

SUMMARY

The responsibilities that the T^2 center must bear if it is to function truly as an agent of technological change and innovation have been examined. North Carolina's T^2 center is beginning to investigate how well it is communicating technological change to local groups. The center's experiences with this task have been presented for review, and suggestions

have been offered to other centers on the basis of its findings so far.

In summary, the following points have been made:

- T^2 centers are faced with three main responsibilities with respect to transferring technology to local agencies: (a) identifying the transportation needs of local officials, (b) packaging the appropriate technology for meeting those needs so that it may be easily understood and implemented, and (c) following up on locals' use of new technology to make sure its implementation is successful (i.e., solves a problem or satisfies a need).
- Although most T^2 centers have learned to accomplish the first two responsibilities effectively, they still need to develop strategies to accomplish the third.
- T^2 centers must learn how locals are using technology presented at workshops and demonstrations if they are to understand fully the success of their technology transfer efforts. This strategy may consist of the following three tasks: (a) creating a climate conducive to technological change; (b) monitoring local attempts to implement new technology; and (c) promoting "local success stories" concerning technological change.
- North Carolina's T^2 center found that revising its course evaluation form enabled it to accomplish the first task and subsequently identify factors that contribute toward establishing a climate more conducive to change, such as support from local roads managers, availability of follow-up training courses, and strategies for funding technological innovation. To accomplish the second task, the center plans to conduct follow-up surveys 3 months after each workshop to determine how locals utilized the technology to which they were introduced once they returned to their work environment. To accomplish the third task, the center's newsletter offers the best opportunity to promote local success stories and foster networking among local agencies which share similar needs.

T^2 centers must regularly communicate with local officials at all stages of the technology transfer process—and this especially includes following up on locals' use of the information and materials presented to them by the T^2 center. Traditionally, North Carolina's T^2 center, like others, has spent its time in the following manner: planning (20 percent); conducting training (70 percent); and follow-up evaluation activities (10 percent). However, if technology transfer is to be successfully carried out at the local level, the following division of time might be more appropriate: planning (20 percent); conducting training (50 percent); and follow-up evaluation activities (30 percent). More effort needs to be devoted to developing program evaluation strategies that allow the center to bear witness to technological change and the benefits emerging from it. Without this knowledge, the T^2 center will remain ignorant of its true contribution to technological change among local agencies.

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The contents of this report reflect the views of the author and do not necessarily reflect the official views or policies of the University of North Carolina or the Institute for Transportation Research and Education.

Planning Model for Rural Roads

ASHOK KUMAR AND H. T. TILLOTSON

Intensive rural road development programs are under way in most of the developing countries but the task of implementing such programs is huge and complex, requiring comprehensive and coordinated planning of all the rural road development activities. The current planning practices are based mainly on ad hoc criteria that are not a good enough basis for planning major schemes. A planning model for systematically generating rural road networks is described. The model can be applied to any country that is planning a rural road network, even though the data used in developing the model belong to India. The model aims at generating a basic rural road network providing road connection from each village to a bigger center of activities for marketing, health, education, trade, social welfare, and so forth. For a rural area, several alternative networks can be generated; each of them will involve different construction costs, maintenance costs, travel costs, and social and economic benefits. The model selects the best alternative by minimizing the total cost or by aiming at trade-offs between the given parameters. An iterative process has been developed for minimization of the total cost of a rural road network. The model is computer based to extend its utility and application, and its data requirements have been limited to what can be available from published census records, local government offices, and existing topographical maps.

In India, rural roads have been a matter of high political and development priority for many years. The government is committed to improving the quality of life in rural areas for which a basic rural road network is considered essential. All aspects of rural development including agriculture, health, education, dairy development, forestry, fisheries, small-scale industries, trade, commerce, and postal services depend on good communication. About 50 percent of the budget for road sector is spent on rural roads. There are about 600,000 villages in India. Intensive rural road development programs are underway to provide all-weather road connections to all these villages by the year 2001 as envisaged in the current plans (*1*).

Even though there is a political will for rural road development, the task of implementing such a rural road development program is huge and complex. Appropriate technologies, trained manpower, suitable organization, and other infrastructure facilities for implementing such a big task are yet to be developed. Apart from this, there are serious constraints on funds. Until now, only about 40 percent of the 600,000 villages are connected with all-weather roads. The remaining villages require about 1.3 million km of new road construction, involving an expenditure of about 400 billion rupees (*1*,*2*). The requirement of maintenance funds is about 30 billion rupees per annum. Currently, only about 10 billion rupees per annum is allocated for new construction of rural roads. Allocation for the maintenance is again only 20 to 30 percent of the actual requirements (*3*).

In addition to the mobilization of additional resources, concerted efforts need to be made for optimal use of the current resources. The first step in this direction is to plan all the rural road development activities in a comprehensive and coordinated manner, taking into consideration the long-range aspects of planning. The present practices of rural road planning leave plenty of scope for further improvements. Because of the absence of a scientific planning approach, several ad hoc approaches such as attaining a particular road density or bringing a village within a specified distance from a road or achieving road connection of a given proportion of villages within different population groups, are resorted to in the plans (*1*,*4*). One of the major guidelines that is now being followed is "to link with all-weather roads all villages with population of 1,500 and above and 50 percent of villages with population between 1,000 and 1,500 by the year 1990."

Although such guidelines appear specific, they in fact fail to offer any direction about the manner in which the targets should be achieved. The costs involved can vary enormously between alternative approaches to the same numerical targets. There is no guidance about which villages (among the targeted villages for road connection) should be connected first and where they should be connected. The design, construction, and maintenance standards are also not mentioned. Much is left to be interpreted locally, which provides scope for wide regional variation.

It is essential that the whole process of rural road development should be approached systematically. The first step in this direction is the preparation of master plans on a regional basis by adopting a scientifically based planning methodology. Only in this way can the funding procedure be streamlined and the expenditure be optimally used. The present study has been concerned with the planning of rural road networks. The aim has been to fill the long-felt need for a suitable methodology for preparing a master plan for rural roads (*5*,*6*).

THE PLANNING MODEL

Purpose

Figure 1 shows a typical map of a rural area. Most of the villages are interconnected by earth tracks. In a way these tracks exhibit the desire lines of travel in the area. Because it will require enormous effort and money to upgrade all these tracks, the proposed planning model aims to provide only the basic minimum road accessibility to villages. The basic road accessibility is defined as an all-weather road connection from each village to a bigger center of activities for marketing, health, education, trade, commerce, and social welfare. Such

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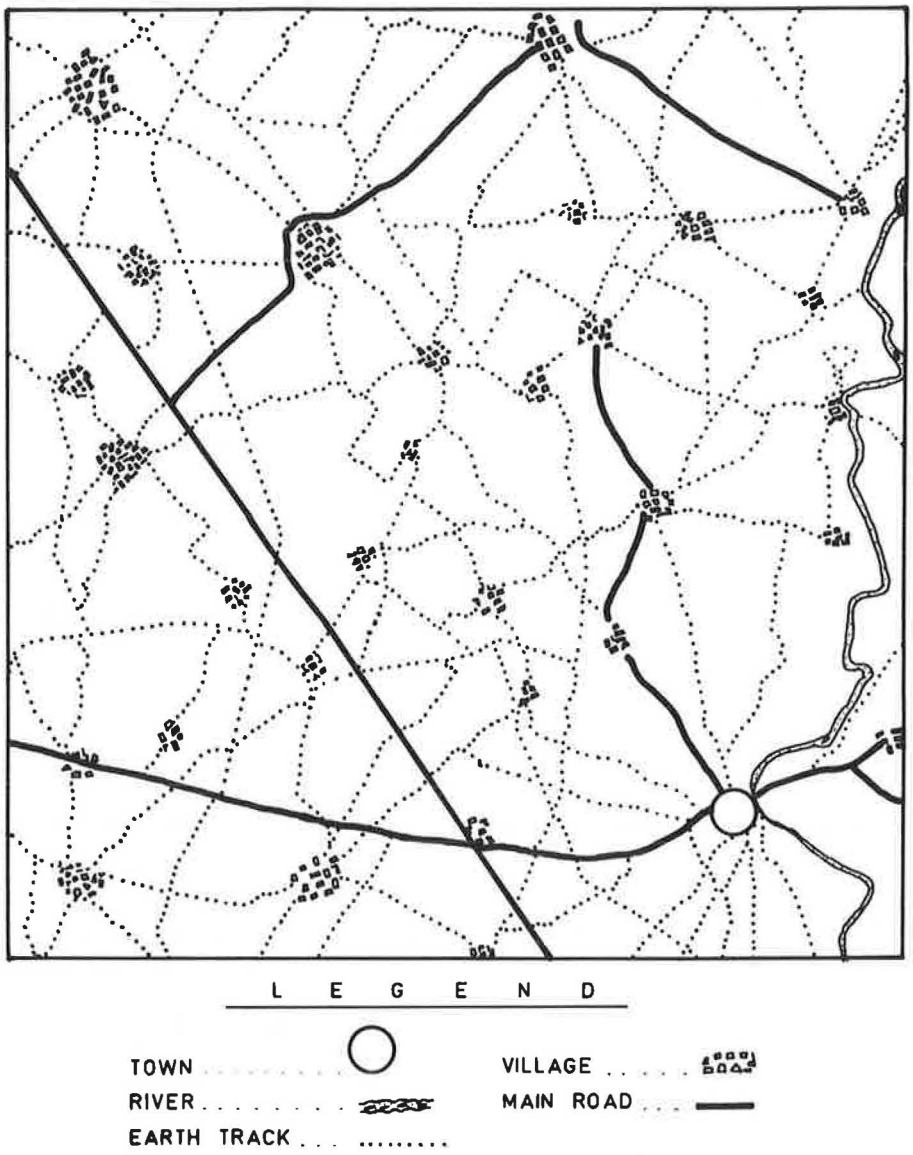

FIGURE 1 Typical map of a rural area.

centers are termed "market centers," which may be situated at the nearby towns, cities, or even at big villages.

It has to be recognized that even after providing all-weather road connections to the villages, the existing tracks interconnecting the villages will play a significant role in rural transportation. The journeys made between a village and farms, and between a village and other nearby villages, will mainly depend on these tracks.

The Optimization Problem

As road access is provided to the villages in a rural area, a large number of alternative networks need to be evaluated on the basis of parameters that include, among other factors, construction costs, maintenance costs, travel costs, and social and economic benefits. The best alternative will be that for which an objective function of these parameters has optimum value, i.e., a maximum or minimum.

Steenbrink (7) suggested a plausible and reasonable objective for a transportation network as the maximization of the positive difference between the benefits and costs to the society, called the "social surplus approach." The benefits of developing a rural road network include increased agricultural production; increased household income and assets; better access to administrative centers like district and block headquarters; increased visits of government officials to rural areas, resulting in better administrative control; availability of better qualified staff for schools, hospitals, and other services in rural areas; reduced sense of isolation among the rural people; increased industrial activities; and better opportunities for employment. On the cost side, there is (a) cost of constructing various rural road links including the cross-drainage works, (b) cost of maintaining the rural road links, (c) travel costs

borne by the users, and (d) external costs such as environmental damage.

It is difficult to define, evaluate, and measure various social and economic benefits of rural roads in quantitative and monetary units. This task requires a good system of indicators for the well being of the rural community along with their relative weight, travel characteristics of different rural communities and the relationship of these characteristics with the type of road accessibility and spatial structure, and a future development plan for the region. Such information is generally not available for the rural areas in India. However, for the purposes of the present model it is the relative benefits of the alternative networks for an area that determine the optimum, rather than the absolute values of benefits. Therefore, several legitimate simplifications can be made to avoid the complex problem of evaluating the social and economic benefits of alternative optimum rural road networks for an area.

For a village, the basic travel demands for general marketing, sale of agricultural and dairy products, procurement of inputs for agriculture, visits to health and education facilities, etc., can be considered inelastic—at least for the alternative networks in the vicinity of an optimum network. Accordingly, little difference in trip patterns and the benefits provided by alternative networks in the vicinity of the optimum may be expected. Hence, the accessibility provided to the villages (or the opportunity provided for travel) can be considered as a whole to be the benefit of a rural road network. Achieving this benefit at the least cost becomes the objective in that case (*7*). Thus, the objective function for minimizing total cost can be expressed as

$$\text{Min } F = m * Z(X,C) + I(C) \quad (1)$$

where

C = dimensions of the rural road links (such as length, design, and construction standards);
X = traffic flows on the rural road links;
$I(C)$ = construction (plus maintenance) costs as a function of dimensions of the rural road links;
$Z(X,C)$ = travel costs as a function of traffic flow and dimensions of the rural road links; and
m = a factor that makes it possible to compare (and add) travel costs and construction costs.

Minimization of investments for a given level of effectiveness (level of service) can also be used as an objective function for optimization of a rural road network as an alternative to minimization of total cost (*6*).

Estimation of Costs

For the present model, the costs have been divided into two types. One type has been called construction cost, which also includes maintenance cost suitably discounted over the life of the project. The model assumes construction cost for a link as proportional to its length, but with the provision of a weighting factor to reflect different construction standards and number and type of cross-drainage works of some links. This weighting factor can be estimated with a reasonable accuracy from the cost data on different type of road construction, cross-drainage works, and labor and material rates available with local road construction agencies.

Another type of cost considered in the model is travel cost. For a rural road link, the travel cost will be the sum of the costs for each trip made over that link. Accordingly, the number of trips generated from each village and their respective costs needs to be known. The number of trips generated by a village will depend on the parameters including population, land use, occupation, income, vehicle ownership and socioeconomic development. The cost for a trip depends on its purpose, length, mode, riding quality of the road, and time involved in making that trip. In addition to conventional motorized vehicles, additional modes need to be taken into account such as bullock carts, bicycle, and pedestrian trips that either do not appear or are of less economic significance in conventional transportation planning followed in the West. Suitable data on trip generation rates from the rural areas in India and on operating costs of various nonmotorized vehicles are not yet available. This limitation has led the model to avoid dependence on the absolute value of travel cost; however, the results obtained by the model are demonstratively worthwhile. For the purpose of the present model, it is the relative costs of alternative road connections for a village, rather than the absolute costs of road connections, which determine the optimum. Therefore, several legitimate simplifications can be made (*6*).

Travel costs for a village can be considered as proportional to (a) the number of persons connected through that village and (b) the distance between that village and the destination of the traffic (the market center). It is therefore argued that whatever the travel costs may be, they will be proportional to a factor in units of person-kilometers, defined as the product of the population connected by a village and the distance between the village and the destination of the traffic. The model is aimed at relatively small uniform areas having similarity in terrain and environmental conditions, agriculture, income, occupation, and habitation patterns. The rural area for which an optimum network is to be generated can first be divided into such small uniform areas bounded by main roads or natural barriers, and the rural road network can be generated for each of these areas. This process restricts the data requirement of the model to the readily available data—like population of villages, coordinates of villages, linear distances between the villages—avoiding carefully any data that are difficult to obtain in rural areas.

The proxy for the construction cost is the length of the link and for the travel cost is the person-kilometer, which are not directly comparable units. Therefore, a conversion factor, RC, has been applied to the link lengths to convert them into equivalent person-kilometers. Hence,

$$\text{Total Cost} = \text{Person-Kilometers} + \text{RC} * \text{Kilometers} \quad (2)$$

The value of RC determines the trade-off between construction and travel costs. A reduction in the number of person-kilometers is a benefit that has to be compared with the cost of extra link-length in kilometers. In the iterative process discussed previously, replacement of one link with another is worthwhile only if the benefits exceed the costs,

that is, if the change in person-kilometers exceeds RC times the change in kilometers.

The value of RC can be estimated by converting the construction, maintenance, and travel costs into equivalent annual costs or net present values. As discussed earlier, the construction and maintenance costs can be estimated easily, but the data required for the estimation of travel costs are not yet available. So the calculation proceeds as follows. The average costs of a trip per passenger per kilometer and goods movement per tonne per kilometer can be obtained from the local transport department or from the passenger and goods transporters. Through sample household surveys in selected villages, the average passenger trip generation rate and goods movement between the villages and market centers can be estimated. RC can then be estimated as follows:

$$RC = CA/(TP * CP + TG * CG) \times 365 \quad (3)$$

where

CA = equivalent annual cost for construction and maintenance of 1 km of a rural road,
TP = average passenger trips per person per day between a village and a market center,
TG = average goods movement per person per day (in tonnes) between a village and a market center,
CP = average cost of a passenger trip per kilometer, and
CG = average cost of goods movement per tonne per kilometer.

As an alternative to determining the value of RC by Equation 3 (because of lack of data), engineering judgment can be used in finding a trade-off between the construction and travel costs. This can be done by generating alternative optimum networks by adopting a set of RC values over a reasonable range. Each of these networks will be optimum for a specific value of RC. The road construction lengths and person-kilometers involved in each alternative optimum network are plotted. Figure 2 shows such a plot for the rural area shown in Figure 3. As is to be expected, when construction costs are at a minimum, small increases in these costs yield large savings in travel costs, but further increases in the construction costs are progressively less productive. The relative benefits of various levels of investments (in terms of road length required for construction of the network) in reducing the travel costs may be seen on the graph in Figure 2. The networks with excessive travel or construction costs, such as in the regions marked *A* and *B*, may be avoided. Engineering judgment will be swayed by the total construction cost if the budgets are constrained but in the knowledge of consequent changes in person-kilometers. It seems reasonable to adopt a network from the knee of the curve in Figure 2 in such

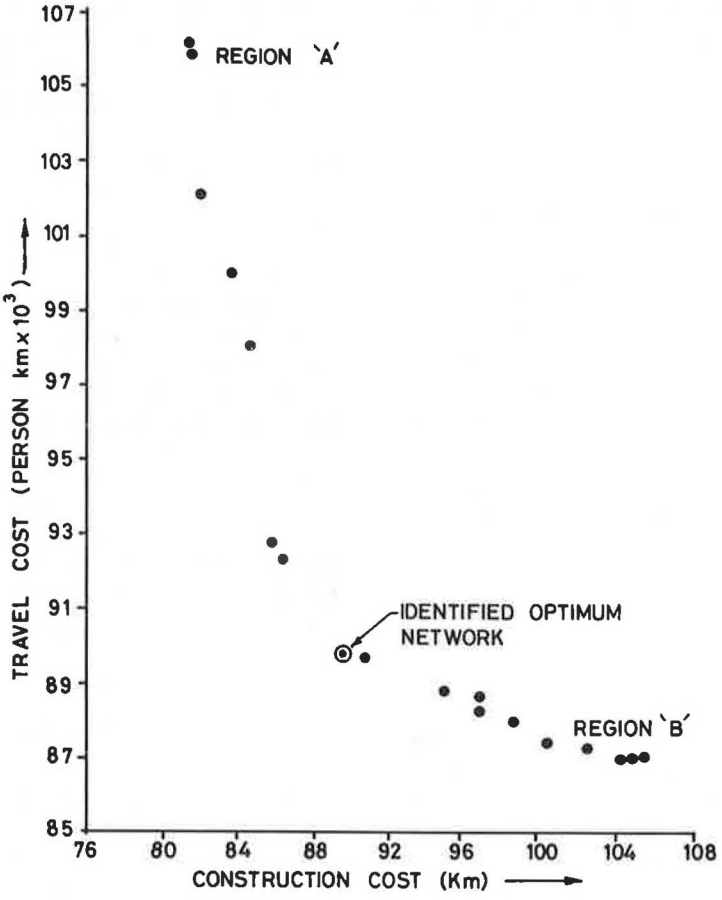

FIGURE 2 Relationship between pseudotravel and pseudoconstruction costs of alternative optimum networks for a typical rural area.

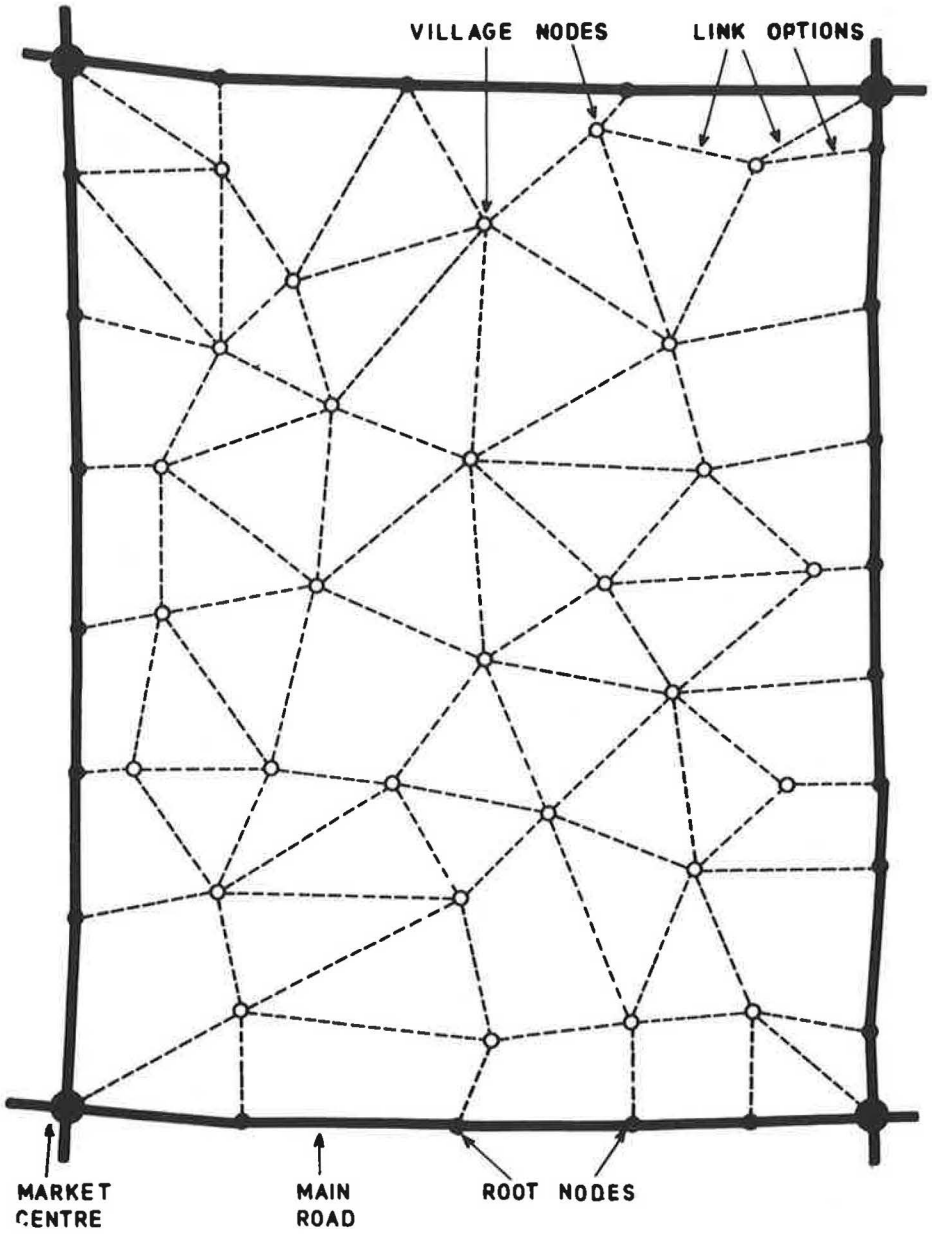

FIGURE 3 A typical set of village nodes, root nodes, and market centers.

cases. This procedure takes advantages of the initial rapid fall of person-kilometers for small extra construction costs.

The Algorithms

The model generates an optimum network for a given set of market centers, main roads, village nodes to be connected, root nodes (lying on main roads) to which the village nodes can be connected, and link options that can be used for connecting the villages (Figure 3). The process of generating the optimum network starts from any rooted tree network, providing one road connection to each village, as shown in Figure 4. A minimum spanning tree that involves minimum link length for connecting the villages can be a good starting network that can be generated by using the standard algorithm of graph theory (*8*). This network will not be regarded optimum by its users because the routes in this network may be quite tortuous, even though each village will have an all-weather route to the main road system. The model makes substitutions in this network in such a way that the sum of construction and travel costs is minimized. An iterative process has been developed by which an inspection is carried out for each village node to examine if the objective function, i.e., total cost, can be reduced by replacing the link connecting that village by other link options available for connecting that village. If so, then the existing link is replaced by the link option providing the lower value of the objective function. By the time the inspection is completed, a new network may be generated.

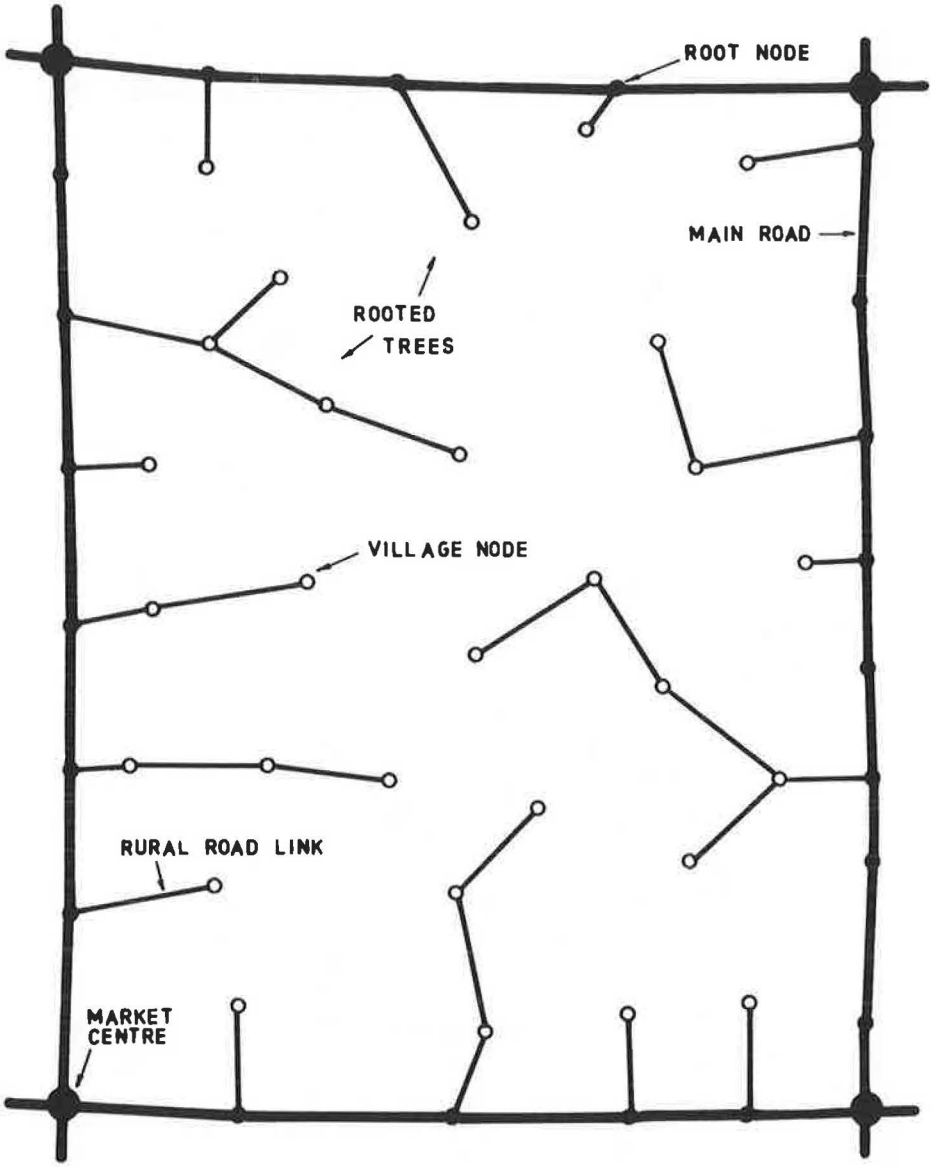

FIGURE 4 A rooted tree network.

The iterative process again starts for this new network and proceeds node by node. The process terminates when it is no longer possible to reduce the value of the objective function.

The algorithm is amenable to solution by a computer and the proposed model is computer based. A computer program is written for the model in FORTRAN 77 language. After generating an optimum rural road network, the computer program categorizes links of the optimum network into three categories (on the basis of population served by a link) for adopting suitable design and construction standards. The program also assigns construction priorities to various rural road links on the basis of population served per kilometer of road length.

Special Considerations for Thinly Populated Areas

In some thinly populated areas like hilly areas, households of the same village may be located far apart, sometimes as far as 7 km. Because of the nature of topography and soil in such areas, suitable land for agriculture and orchards (or even for the construction of houses) is available in small scattered bits, and villagers have a tendency to construct their houses close to the farm lands. The concept of a village formed by a group of closely located houses (as found in most rural areas) may often not exist in these areas. In such cases, attention needs to be focused on opening up the entire inhabited area of a village rather than connecting a specific node. In such situations, the model can be applied as follows:

1. Identify the inhabited areas of each village and locate such model points that, if connected by all-weather roads, can bring most of the inhabited areas within a reasonable distance from an all-weather road.
2. Identify various feasible alignments of road links interconnecting these nodal points, which can be developed to all-weather standards. Minor adjustments in location of the nodal

points, to suit engineering and geological requirements of these links may be necessary.

3. Through these steps, a set of nodes and link options can be arrived at, for which an optimum rural road network can be developed.

Incorporating Existing Rural Road Network in the Optimum Network

In most rural areas, some part of the rural road network may already be constructed, which would need to be considered during generation of an optimum network for that area. This could be done by making a suitable allowance in the construction cost of link options already constructed.

For the links already constructed, the construction cost may be treated as a part of the actual construction cost or just the cost of road maintenance alone. Alternatively, a rural road network can be generated without any regard to existing rural road network, in the first instance. Thereafter, the existing rural road network can be superimposed over the optimum network. This superimposition can then form the basis of any modification in the optimum network that can be justified because of the roads already constructed.

Different Applications of the Model

The model can be effectively used for preparing master plans for rural roads. The model can provide a uniform basis for rural road planning. Currently, planning practices differ widely from state to state and are swayed by local judgment.

Alternative combinations of market centers, main roads, and villages for an area can be evaluated by using the model. The model can also be applied for the identification of major bridge sites in rural areas. Even though major bridges are not anticipated on rural roads, the provision of a major bridge may emerge as the best alternative in some rural areas lying across a river or stream. In such situations, alternative rural road networks for the area can be generated with and without the provision of major bridges at feasible locations, and the alternative giving least cost can be adopted.

A direct estimate of the requirement for rural roads for each district and state and the country as a whole can be made by using the model. The task can be performed by obtaining the population of villages from census records and the coordinates of various villages and root nodes from the existing topographical maps. The coordinates can be directly measured from topographical maps and fed to a computer. Simultaneous with the requirement of rural roads, the need for establishing new market centers and developing main roads can also be assessed. The actual requirement for rural roads for a region or state, when compared to the existing rural roads in that region or state, can become a strong basis for allocating funds for rural road development among the various regions.

EXAMPLES

The model was applied to a part of a district shown in Figure 5. In this district, all the villages were already connected with rural roads. This made it possible to compare the existing rural road network and the optimum network generated by

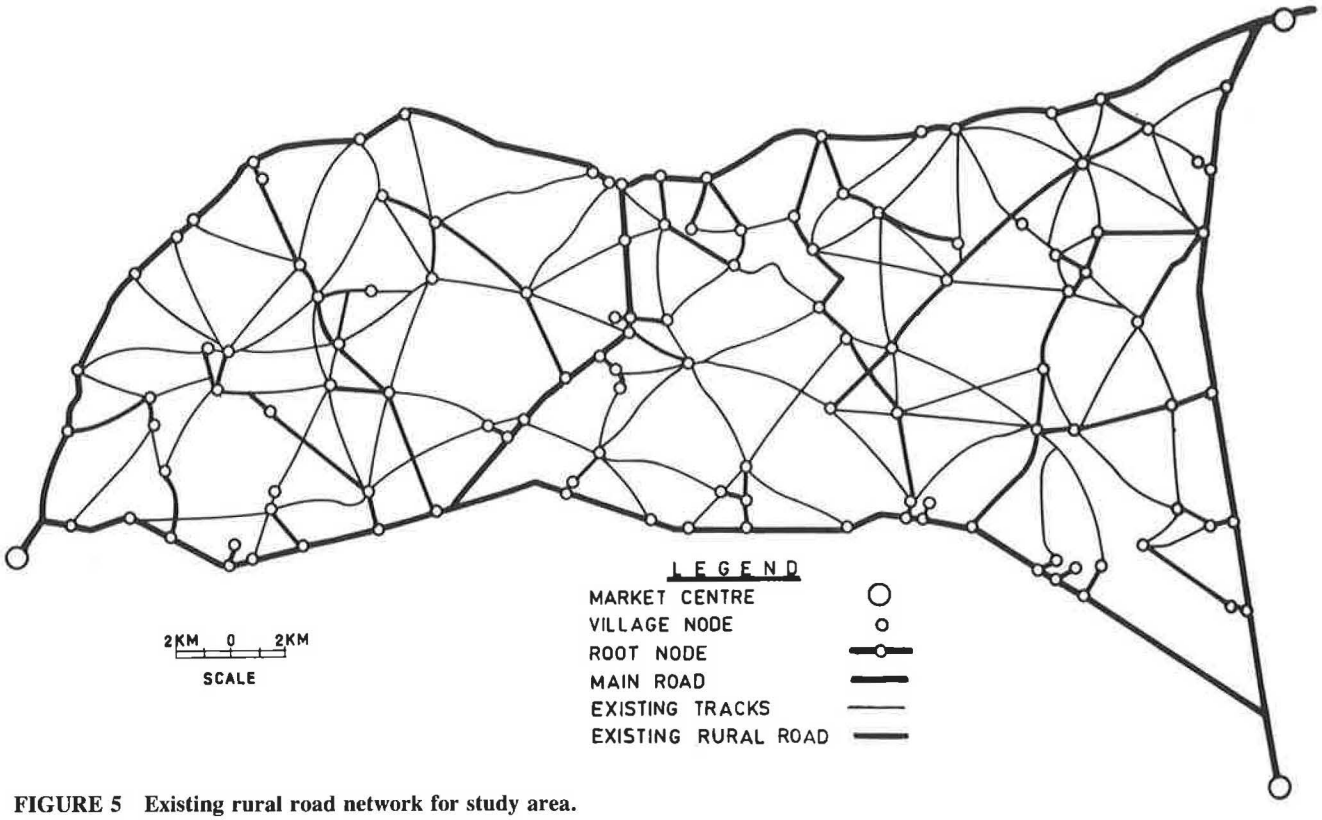

FIGURE 5 Existing rural road network for study area.

the proposed model. The optimum network was generated by aiming at a trade-off between travel costs and construction costs as explained previously. A set of 32 alternative optimum networks was generated by adopting different RC values ranging from 100 to 30,000. The pseudoconstruction and pscudotravel costs (road lengths and person-kilometers) of the alternative optimum networks are shown in Figure 6. Rapid fall in the travel costs with small increases in construction costs can be observed up to an RC value of 1,400. Accordingly, the network generated with an RC value of 1,400 was identified as the optimum, providing a reasonable trade-off between the construction and travel costs. The optimum network thus identified is shown in Figure 7. Table 1 presents a comparison between the existing and the optimum network.

The existing network is costlier to construct as well as costlier to the users. The existing network requires 157.58 km of road length for providing one road connection to each village, whereas the optimum network requires only 144.2 km of road length to connect the villages. Pseudotravel cost for the existing network is 1 560 278 person-kilometers, whereas for the optimum network it is only 1 401 981 person-kilometers. The extra road length of the existing network does not show any benefit in terms of reduction in travel cost. On the contrary, the extra road length in the existing network makes the network less efficient. It is interesting to note that even with lower road length the optimum network provides far more efficient road connections to the villages. Efficiency for a village node in the optimum network was calculated as

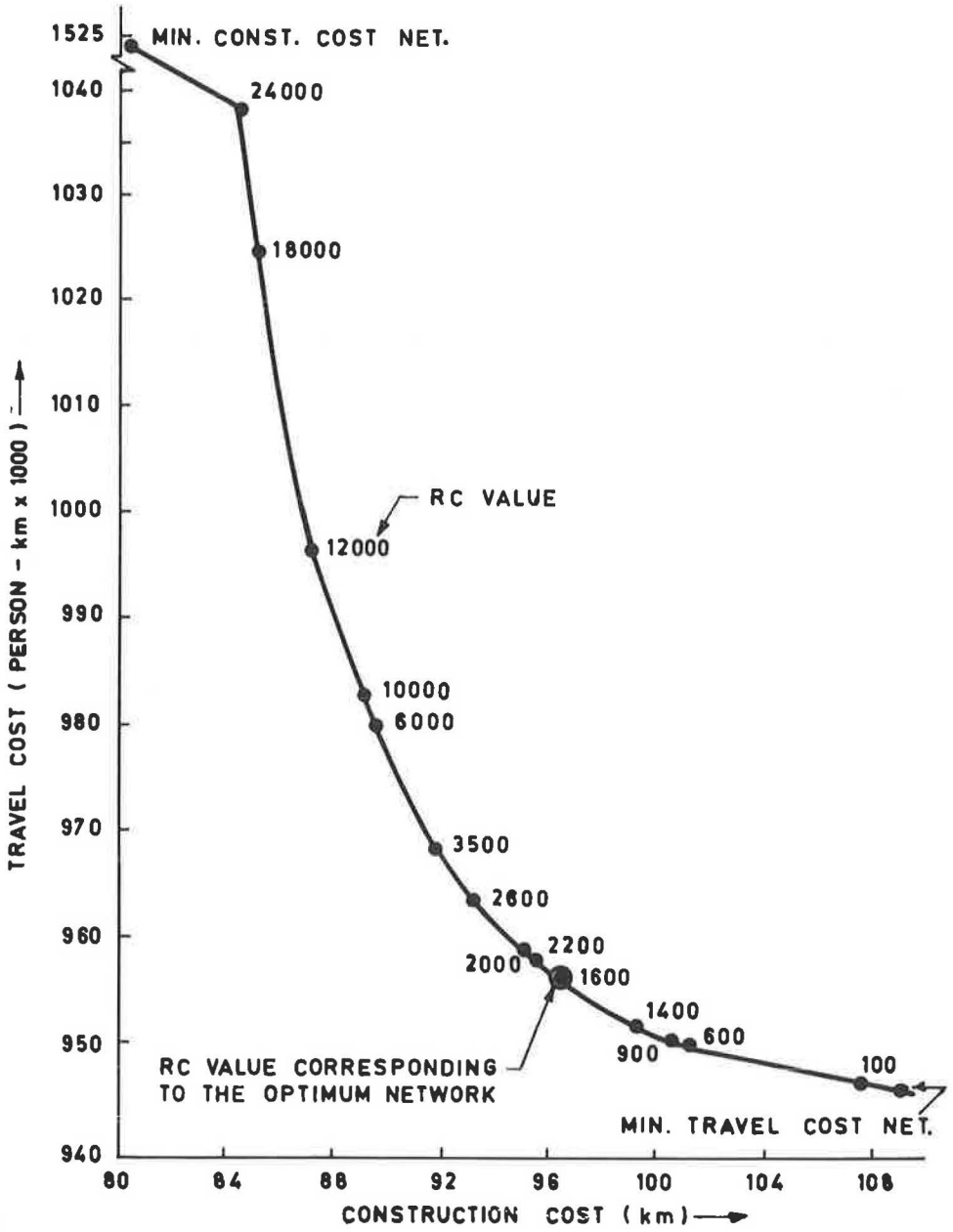

FIGURE 6 Pseudotravel and pseudoconstruction costs of alternative optimum networks.

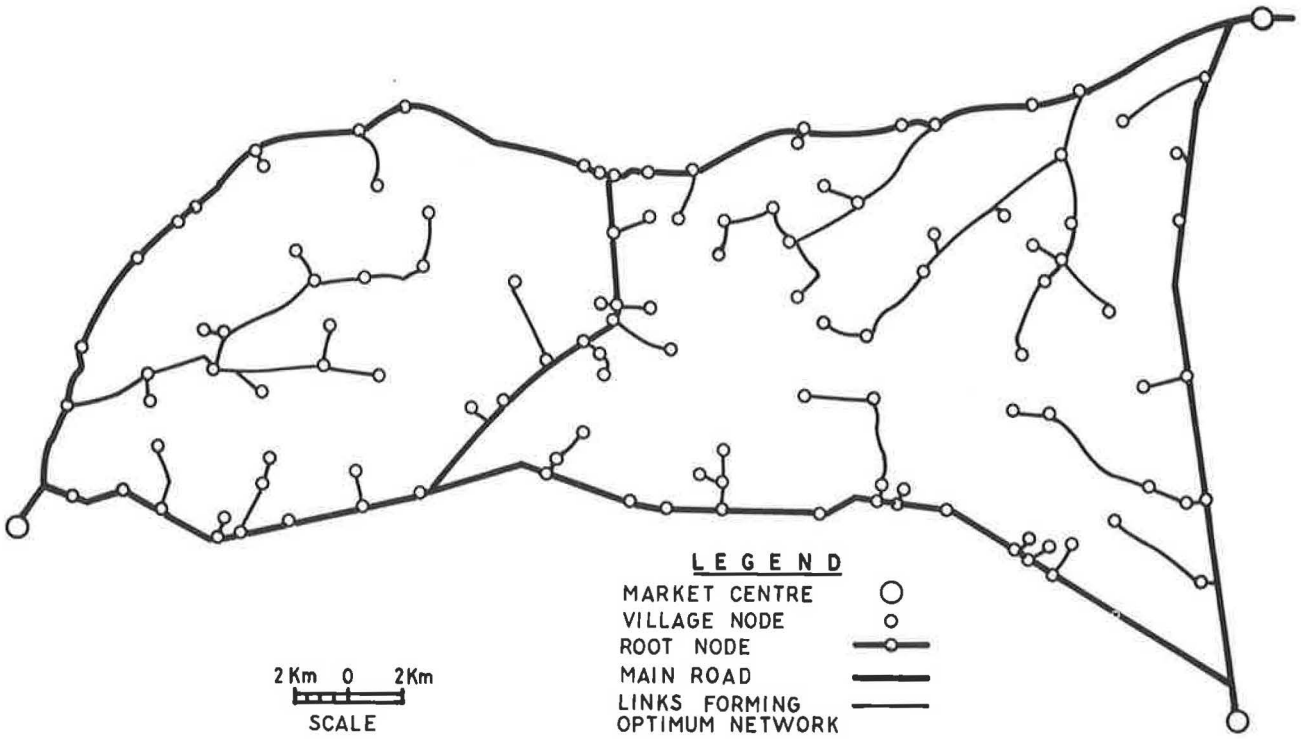

FIGURE 7 Optimum rural road network for study area.

follows:

$$EO = \frac{DMIN}{DACT} * 100 \quad (4)$$

where

EO = efficiency of a village node in an optimum network,
DMIN = distance of the village node from the destination in the minimum travel distance network, and
DACT = distance of the village node from the destination in the optimum network.

The model was also applied for the evaluation of two alternative spatial structures for generating the rural road network between rivers *R1* and *R2* (Figure 8). In the first alternative, a main road was to be constructed along the canal flowing between rivers *R1* and *R2*. The villages in the area can then be connected to this road through the rural road network. In this way, the villages can be connected to the market centers *M1* and *M3*. In the second alternative, main roads *A*, *B*, *C*, and *D*, and four cross-drainage works on River *R2* as shown in Figure 8 are to be constructed (other bridges shown in Figure 8 already exist). The villages can be connected to these main roads through rural road networks. Rural road networks were generated for both of these alternatives and the related

TABLE 1 COMPARISON BETWEEN THE EXISTING NETWORK AND THE OPTIMUM NETWORK FOR THE STUDY AREA

SN	Parameter	Existing Network	Optimum Network
1	Total Person-km	1,560,278	1,401,981
2	Pseudo Construction Cost (km)	157.58	144.24
3	Average Efficiency (%)	88.43	98.39
4	Average Efficiency for Villages of Differnent Population Category		
	a) Less Than 500	88.06	96.44
	b) Between 501-1000	84.16	98.90
	c) Between 1001-1500	87.57	98.65
	d) Over 1500	92.78	98.32

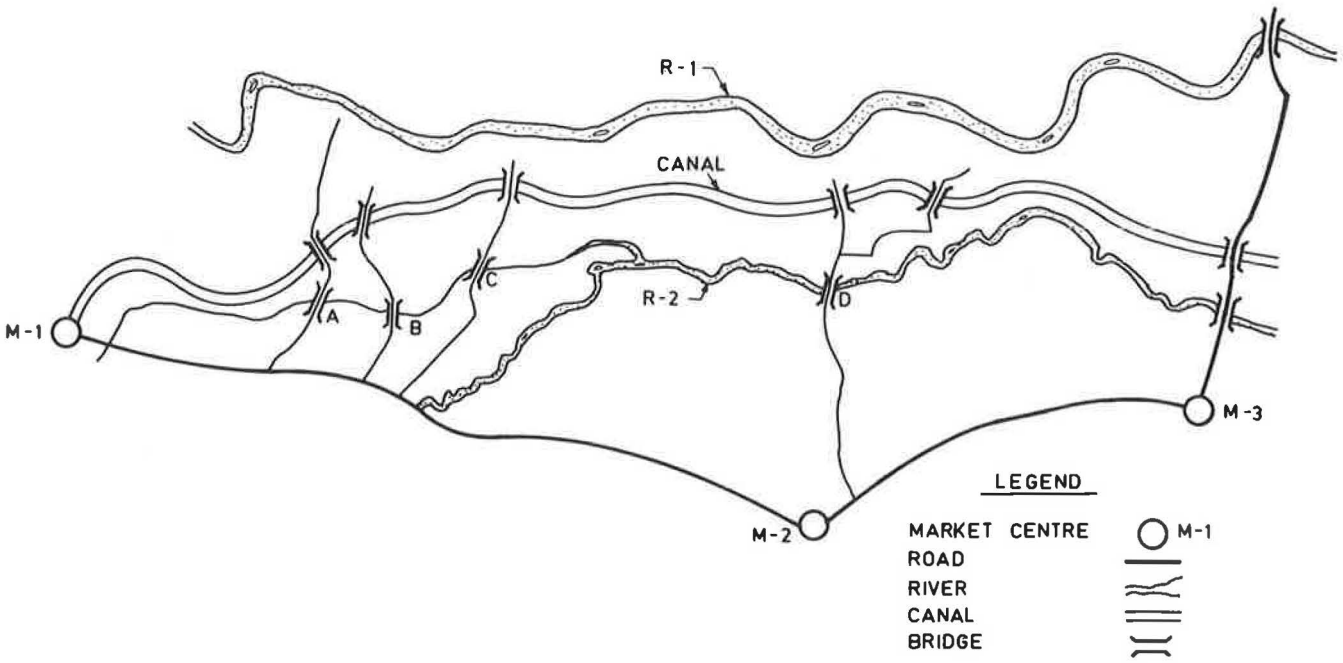

FIGURE 8 Evaluation of alternative spatial structures.

costs are presented in Table 2. The second alternative was found to be much cheaper in terms of construction cost as well as travel cost.

LIMITATIONS AND FURTHER RESEARCH WORK

The model provides only the basic minimum accessibility to a village by connecting that village to a market center or to a main road. The trips from a village to other villages and to market centers other than the one to which that village is connected in the optimum network are ignored.

Through routes can be provided by connecting dead ends of various rural roads in the optimum network generated by the model. The through routes can be provided by adding only a few links in the optimum network. Such through routes will be useful for bus services, milk collection services, and other mobile facilities for health and marketing, providing economy and operational convenience.

The model assumes fixed passenger travel demand and fixed passenger travel cost per kilometer for evaluating the travel costs, which may not be so in reality. The passenger travel demand will depend on the patterns of employment, land use, occupation, income, services, and expectations. The model assumes these patterns to be similar for the group of villages considered for generating the optimum network. The cost of travel per passenger per kilometer will depend on vehicle route structure, passenger loading, mode and length of the trip, and road surface type. The model can be further refined

TABLE 2 COMPARISON BETWEEN THE ALTERNATIVE SPATIAL STRUCTURES

SN	Item	Alternative I	Alternative II
1.	Construction Cost (million Rs.)	42.15	31.63
2.	Pseudo Travel Cost		
	(i) Person-Km	2,380,000	1,400,000
	(ii) Tonne-Km	3,490,000	2,000,000

Note:

(i) The construction costs were calculated by multiplying the road length to be constructed with the unit rate of road construction.

(ii) Tonne-Km were calculated by estimating the quantum of agricultural produce to be transported from each village.

by taking into account the variation in passenger travel demand and passenger travel cost.

CONCLUDING REMARKS

A planning model has been developed that generates a basic rural road network providing access to each village to a larger center of activities for marketing, health, education, trade, commerce, and social welfare. The planning model will be useful to the planning authorities for preparing master plans for rural roads. The planning model can provide a systematic and uniform basis for planning of rural roads, apart from providing economy in the rural road development.

ACKNOWLEDGMENTS

The authors are grateful to D. V. Singh, Central Road Research Institute (CRRI), New Delhi, India, and to Y. R. Phull and A. V. S. R. Murty of CRRI for their suggestions and support for this work.

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Sign Advisory System for Low-Volume Roads

FONG L. OU, DAVE BADGER, AND BRIAN LESSER

Development of a prototype expert system for signing low-volume roads is described. The system consists of the following four modules: (a) basic sign training and overview, (b) determining the need for signs, (c) determining the size of guide signs, and (d) sign placement. The system is intended to be used as a training tool for personnel with limited traffic control device design or placement experience. It is also intended to be used as one resource to aid in the selection, procurement, placement, and installation of appropriate road traffic control devices. The system was evaluated by the Forest Service field units with favorable response. It was also felt that the system may be useful to other federal and county agencies. Therefore, the system was submitted and subsequently approved as a Coordinated Federal Lands Highways Technology Improving Program (CTIP) project. The Forest Service is currently working with the CTIP agencies and National Association of County Engineers in the system development effort.

This paper describes the development of a prototype expert system for signing low-volume roads. The prototype system was developed for demonstration purposes and is not ready for implementation. The system consists of four modules: basic sign training and overview, determining the need for signs, determining the size of guide signs, and sign placement.

Many government agencies do not have traffic engineering expertise available wherever and whenever needed because of personnel and budget limitations. At times, the planning, design, and installation of traffic control devices are the responsibilities of personnel who are not trained or experienced in traffic engineering or the application of the *Manual on Uniform Traffic Control Devices* (MUTCD) (*1*). This situation increases the potential for incorrect installation of traffic control devices and other signs, and may lead to unsafe conditions and other undesirable results, including increased costs.

In the Forest Service, many experienced personnel with technical sign expertise are approaching retirement. To retain their expertise, the Forest Service decided to develop a road sign advisory expert system for use as a training tool for personnel with limited traffic control device knowledge and experience. The system may also be used to aid in the selection, procurement, placement, and installation of appropriate traffic control devices.

The developed prototype system contains a first-stage module for personal computers using artificial intelligence. This module addresses warning signs pertaining to horizontal alignment. The artificial intelligence system queries the user about site and traffic conditions and interactively provides written and visual guidance for proper sign selection, sizing, and installation on the basis of published standards.

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The module was evaluated by Forest Service field units. Responses were favorable and supportive of further module development. Initial indications were that the system could provide needed training and also serve as a user-friendly signing information resource for those not skilled in traffic engineering. In May 1990, the Road Sign Advisory Expert System was selected as a Coordinated Federal Highways Technology Improvement Program (CTIP) project. The Forest Service is working with the CTIP agencies and National Association of County Engineers (NACE) to expand the prototype system to cover the whole spectrum of warning signs and selected regulatory signs (i.e., Stop, Yield, and Speed Limit) for conventional and special purpose roads. Other sign categories will be included following completion of this work. The CTIP agencies include Federal Highway Administration, Bureau of Land Management, National Park Service, Forest Service, and Bureau of Indian Affairs. This development work is expected to be completed by the end of 1991.

THE MAKEUP OF THE EXPERT SYSTEM

An expert system is a computer program that performs at the level of a human expert in a complex but narrow field and uses human traits such as logic to solve problems. It is composed of a knowledge base and an inference engine. The knowledge base is made up of data and rules by which conclusions are reached. The data may be either factual, heuristic, or empirical. The inference engine is the problem-solving component of the expert system. It includes a collection of processing procedures for examining data-using rules.

Software and Hardware

IBIS (*2*), a commercial system shell for developing expert systems, was used for this sign advisory system. It provides the framework for building expert systems in the same manner as templates are built for accounting spreadsheet programs. Hardware requirements for IBIS are IBM-compatible MS–DOS machines with 512K ram memory and printer. A DOS version 2.0 or higher is required. Printer may be either HP Laserjet or Epson compatible.

Knowledge Base

Two different types of knowledge used in engineering practice are deterministic and heuristic. Deterministic knowledge is

preserved in references, manuals, handbooks, and textbooks. Heuristic knowledge is the subject matter knowledge possessed by individuals, characterized by beliefs, opinions, experience, and rules of thumb.

Most of the knowledge base data used for this system is deterministic knowledge obtained from MUTCD (*1*), *Manual of Traffic Engineering Studies* (*3*), and Forest Service handbooks (*4*). Heuristic knowledge was obtained from several Forest Service engineers and technicians with road signing experience. Much of the heuristic knowledge was related to determining the need for signs.

SYSTEM DEVELOPMENT

The long-term objective is to develop an expert system that will eventually include the following sign categories and other traffic control devices:

1. Regulatory signs,
2. Warning signs,
3. Guide signs,
4. Construction and maintenance signs,
5. Motorist services signs,
6. Recreation and cultural interest area signs,
7. Tourist-oriented directional signs, and
8. Markings.

Initial development was limited to those warning signs related to changes in horizontal alignment.

The system's four modules are (a) basic sign training and overview, (b) determining the need for signs, (c) determining the size of guide signs, and (d) sign placement (Figure 1).

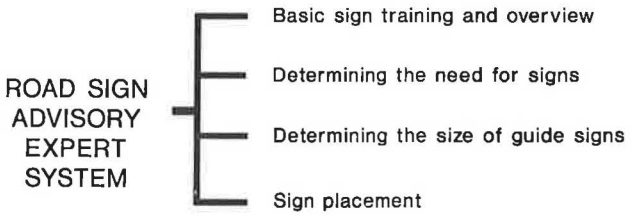

FIGURE 1 Structure of the road sign advisory expert system.

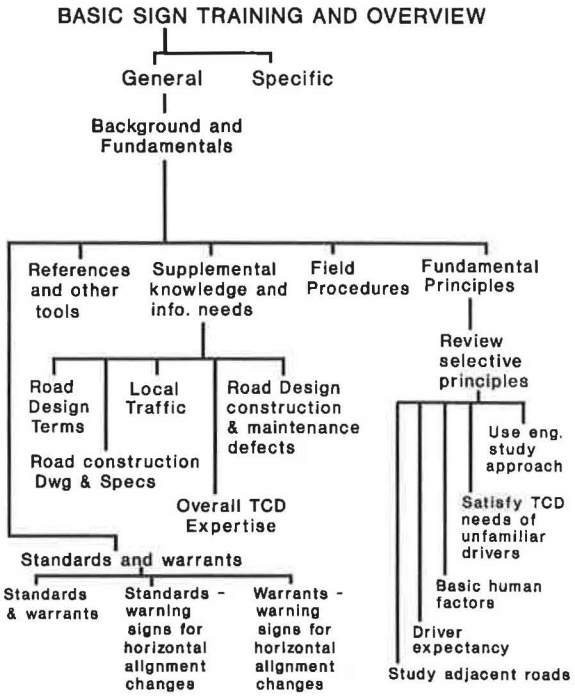

FIGURE 2 General information of basic sign training and overview.

Module 1—Basic Sign Training and Overview

Information included in this module is both general and specific. General information includes: background and fundamentals of sign engineering; basic concepts of signing described in MUTCD, the Traffic Control Device Handbook, and other traffic sign directives; the definition of traffic control devices; and so forth. Figure 2 shows a sample of general information contained in Module 1.

Specific information describes the use and detailed characteristics of individual warning signs. As shown in Figure 3, warning signs were classified into eight categories:

- Advisory speed plate,
- Hazard markings,
- Changes in horizontal alignment,

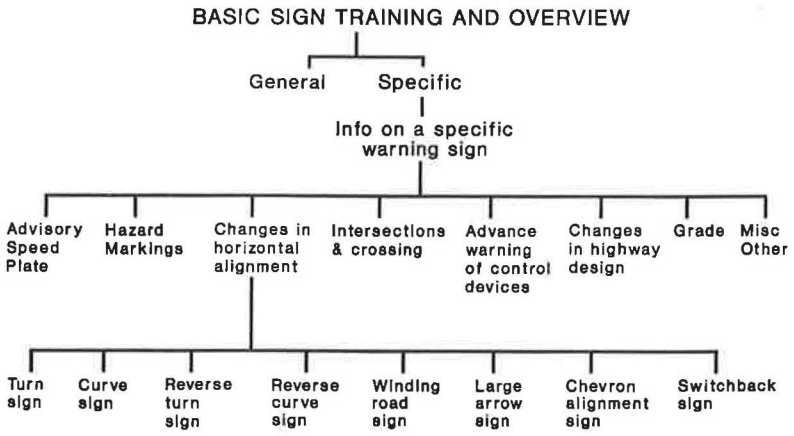

FIGURE 3 Specific information of basic sign training and overview.

- Intersections and crossings,
- Advance warning of control devices,
- Changes in highway design,
- Grade, and
- Miscellaneous.

Individual signs included in the changes in horizontal alignment category are as follows:

- Turn sign,
- Curve sign,
- Reverse turn sign,
- Reverse curve sign,
- Winding road sign,
- Large arrow sign,
- Chevron alignment sign, and
- Switchback sign.

The definition, warrant, and graphic display of each sign may be queried by the user.

Module 2—Determining the Need for Signs

The criteria used for determining the need for signs varies from one sign to another. Factors considered in the selection of a warning sign for changes in horizontal alignment include the vehicle approach speed (V'), the recommended vehicle speed through the change in horizontal alignment (V), the speed differential between the approach and recommended speeds ($\bar{v} = V' - V$), sight distance, driver expectancy, accident history, and the characteristics of road geometry.

The approaching speed denotes the average vehicle operating speed (85th percentile) on a road segment immediately before a change in horizontal alignment. The recommended vehicle speed is defined as the maximum safe speed at which a vehicle can negotiate a change in horizontal alignment. Curve characteristics such as radius, superelevation, and side friction are some factors that affect this value. The recommended vehicle speed may be determined by either mathematical computation or by field test.

The formula for computing the recommended speed is (*5*)

$$V^2 = 15R(e + f)$$

where

V = recommended speed (mph),
R = radius of curve (ft),
e = rate of superelevation (ft/ft), and
f = safe coefficient of side friction.

The recommended speed can also be determined by field test using a ball-bank indicator (*5*). A ball-bank indicator consists of a steel ball in a sealed curved tube. It is mounted on a test vehicle. When the test vehicle is driven around the curve at various speeds, the ball-bank readings provide a measure of the centrifugal force acting on the vehicle.

The prototype program uses the following logic to determine the need for specific signs.

Turn Sign (W1–1)

The recommended speed is 30 mph or less.

Other factors influencing the selection are considered as follows:

1. If the speed differential is 10 mph or greater, a sign is recommended;
2. If the speed differential is equal to or greater than the speed differential used by other local (within a 3-mi range from the road segment under study) road management agencies to determine the need for a turn or curve sign, a sign is recommended;
3. If the sight distance is obstructed from clear view and the speed differential is equal to or greater than 5 mph, a sign is recommended;
4. If the approach grade is equal to or greater than −6 percent and the speed differential is equal to or greater than 5 mph, a sign is recommended; and
5. If the site of interest has a history of previous accidents, a sign should be considered.

Curve Sign (W1–2)

The recommended speed is greater than 30 mph.

The logic for determining the need for curve signs is similar to that used for turn signs.

Reverse Turn Sign (W1–3)

A Reverse Turn sign is recommended for marking two turns or a turn and a curve in opposite directions, as defined in the warrants for turn and curve signs, that are separated by a tangent of less than 600 ft.

Reverse Curve Sign (W1–4)

A Reverse Curve sign is recommended for marking two curves in opposite directions, as defined in the warrants for curve signs, that are separated by a tangent of less than 600 ft.

Winding Road Sign (W1–5)

A Winding Road sign is recommended for marking three or more turns or curves, as defined in the warrants for turn and curve signs, that are separated by tangents of less than 600 ft. If continuous winding road conditions extend over 1 mi, a supplemental distance plaque is recommended. Repeating the distance plaque at two increments is recommended when the average vehicle speed exceeds 35 mph or the winding road conditions exceed 5 mi in length.

Large Arrow Sign (W1–6, W1–7)

A Large Arrow sign is recommended for marking sharp changes in road alignment in the direction of travel.

Chevron Alignment Sign (W1–8)

A Chevron Alignment sign is recommended for marking those sharp changes in road alignment in the direction of travel that require additional emphasis and guidance in addition to that provided by the Large Arrow sign or standard delineators.

Switchback Sign (FW1–U)

A Switchback sign is recommended when (*4*)

1. The vehicle approach speed is 30 mph or less,
2. The deflection angle between the tangents to the curve equals or exceeds 120 degrees, or
3. The speed differential meets the requirements for a Turn sign.

Advisory Speed Plate (W13–1)

The use of the advisory speed plates is recommended under the following conditions:

1. The speed differential is 10 mph or greater and the approach grade is less than −6 percent.
2. The speed differential is 5 mph or greater and the approach grade is greater than −6 percent.

Module 3–Determining the Size of Guide Signs

This module was designed to calculate guide sign panel length and height from the user's input of letter height, number of lines on the sign, and content of each line. It also calculates the area of the sign panel. The criteria used for this module are as follows:

1. Letter, numeral, and spacing widths are based on Series C of the *Standard Alphabets for Highway Signs* (*6*);
2. The spacing for top, bottom, and end margins equals ¾ of the capital letter height;
3. The spacing between lines equals ½ of the capital letter height;
4. The spacing between words and characters equals 4 times the letter stroke width;
5. The fraction length equals 1½ times the capital letter height; and
6. The horizontal arrow length equals 1¼ times the letter height and the vertical arrow length equals the letter height.

Module 4—Sign Placement

This module has not yet been developed. The MUTCD (*1*) and other related references will be used as the major sources of information when it is developed.

SYSTEM OPERATION

System operation starts with a main menu from which the user may select one of the four modules. As shown in Figure 4, the operation algorithm allows the user to repeatedly acquire the information stored in the same level of the data tree or to return to the main menu.

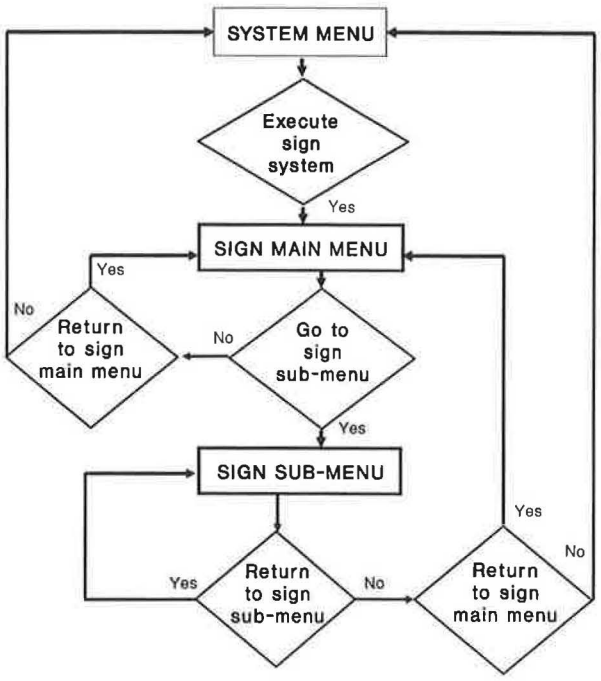

FIGURE 4 Operation algorithm of training and overview.

Module 1—Basic Sign Training and Overview

If a user selects Module 1, "Basic Sign Training and Overview," from the main menu, the system will display a submenu on the lower half of the screen with a question asking, "Which type of information do you wish to review?" There are three choices: general information, specific information, and return to main menu. If the user selects general information, another submenu will be shown on the screen identifying the types of information available for review. If the user selects specific information, the system will display a submenu of individual signs. After the user selects a specific sign, the system will display text describing the sign and its usage along with a graphic display of the sign. If the user selects return to main menu, the user is returned to main menu to select either general information or specific information.

Module 2—Determining the Need for Signs

In order to determine the need for signs for a specific situation, the user first selects Module 2, "Determining the Need for Signs," from the main menu. The user must then select the appropriate sign category from the first submenu and the appropriate sign type from the second level submenu. The user is then led through a series of questions pertaining to

- The number of turns and curves being evaluated,
- The length of tangent between each turn or curve,

- The approaching speed,
- The recommended speed,
- The speed differential,
- The approaching grade,
- Any site obstructions,
- Any accident records for the site under investigation,
- Local agency practice for signing similar situations, and
- Driver expectancy.

On the basis of the user's answers to these questions, the system will make specific signing recommendations.

The system provides four options for the user to display the program output:

- Display;
- Display and print;
- Display and file; and
- Display, print, and file.

For other than the Display option, the user must provide information to identify the location evaluated. This information includes road number, milepost, intersection, sign number, date, region, national forest, and ranger district.

The output report includes location identification information, the conditions on which the recommendation is based, and the recommended alternative. It also provides space for comments and signature. A sample report format is shown in Figure 5 (*left*); a sample graphic display of the recommended signs is shown in Figure 5 (*right*).

Module 3—Determining the Size of Guide Signs

The user may determine the size of individual guide signs by selecting Module 3. On the basis of the user's answers to a series of queries, the sign size will be determined and displayed on the screen. If desired, the user may invoke an EDIT

RECOMMENDED ROAD SIGN

Road No.: 1234
Milepost: 5.2
Intersection:
Sign No.: 100

Date: 3/4/90
Region: R05
National Forest: F10
District: D5

BASE CONDITIONS:

- The approaching speed is 35 mph
- The recommended speed is 25 mph
- The speed differential is 10 mph
- The speed differential used to determine the need for the sign for road segments near the study site is unknown
- The sight distance is obstructed
- The negative grade is less than 6%
- The curve is sharper and less obvious than the normal situation, that is, the change in alignment violates the driver's expectancy
- The I–angle is less than 120 degrees
- No accidents have been reported

The selected alternative: A TURN SIGN WITH CHEVRON ALIGNMENT SIGN AND SPEED PLATE IS RECOMMENDED

Comments – Rationale for not adopting the system recommendation or for selecting another alternative:

Recommended: _____ (signature) (date)

Approved: _____ (signature) (date)

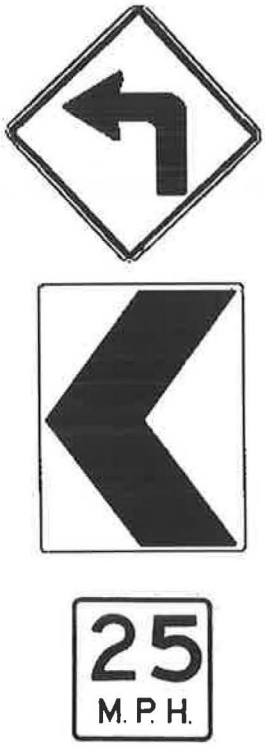

FIGURE 5 (*left*) **Report of sign evaluation and** (*right*) **recommended signs.**

command to adjust the text of the sign by inserting new text, by deleting text, or by rearranging the text or sign layout, or both. Modifications of this type require that the user rerun the program to recalculate the line lengths and sign panel dimensions. At this time, the user may also add information (road number, station, etc.) to identify sign location. When the sign design has been completed, the user may either PRINT or SAVE the output. Note that neither the screen display nor the printed output are to scale.

System queries and user responses for a typical situation are as follows:

System Queries	*Responses*
Please enter letter height in inches	5
Please enter number of lines on the sign	2
Please enter text of line #1	ALBUQUERQUE 5 >
Please enter text of line #2	PECOS 51 >
Do you want line totals? (Y/N)	Y
Do you want the area of the sign panel? (Y/N)	Y

For these responses, the output shown in Figure 6 will be displayed on the screen.

SYSTEM EVALUATION

In May 1989, the prototype of the Road Sign Advisory Expert System and a questionnaire (Figure 7) were mailed to the nine Forest Service regional offices for evaluation. All regions responded, with six of the nine favoring additional developmental effort. Some regions expressed concern that traffic control device decisions required professional judgment that could not be recognized or applied by expert systems.

Most of the users who tested the system indicated that it was easy to use. Some suggestions were made to improve the system to make it more understandable and user friendly. The majority of the respondents indicated that the system is a useful tool for training personnel with limited signing experience. On a 1-to-10 scale, where 1 represents little usefulness and 10 denotes great usefulness, the rating for using the system as a training tool ranged from 5 to 9, with an average of 7.5. The rating for using the system to determine the need for signs ranged from 1 to 10, with an average of 6.6. The majority of respondents would like the system expanded to include signing for other situations such as recreational developments, trails, forest entrance, and administrative sites.

Many respondents agreed that the system could save time; however, they could not determine exactly how much time would be saved.

CONCLUSION

The prototype Road Sign Advisory Expert System presented in this paper demonstrates the feasibility of using expert systems as one tool for managing road signing programs. The system may also be useful as a training aid.

ACKNOWLEDGMENT

The authors thank other project team members—Margaret Bro, Dave Neeley, Doc Morrison, and Milt Taylor—for their contributions to the development of the knowledge base. Special thanks go to Ted Zealley and Jerry Bowser for their intellectual support. We also acknowledge that the program for calculating the size of guide signs was written by B. Wahl and Earl Williams.

This material was developed, written, and prepared by employees of the U.S. government; therefore, it is in the

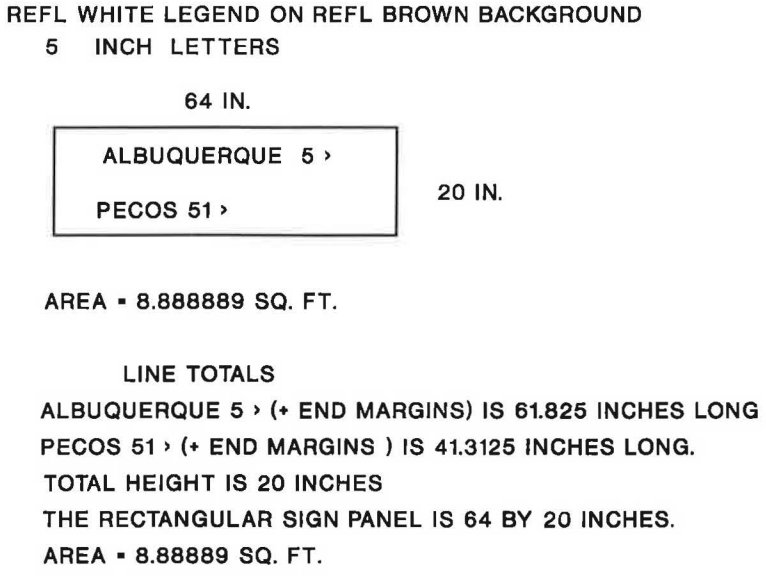

FIGURE 6 Report of calculating a guide sign.

Region _____ Forest _____ District _____

Job Title _____

Level of Sign Expertise: No( ) Low( ) Medium( ) High( )

We Need Your Help!

An "expert" sign model for road signs is proposed for microcomputer use (or DG when the DG version of IBIS, the expert system shell, is available). It would have two purposes: (1) as a training tool for the new or untrained sign coordinators on a District or Forest; and (2) to provide "expert" guidance to journeyman signs persons when deciding whether signs are needed and, where needed, what signs to use.

This prototype model is being developed only for the standard road warning signs relating to changes in horizontal alignment. To become a fully functional system for all road signs, the remaining warning signs and all of the regulatory and guide signs would need to be added.

By answering the following questions, you can help determine the future of this effort:

1. Was the model easy to use? Yes ( ) No ( ) If not, why?

2. What would improve it for the ease of the user?

3. Was the model understandable? Yes ( ) No ( ) If not, why?

4. What would make it more understandable?

5. Did you learn about the change in horizontal alignment signs from it? Yes ( ) No ( ) Explain please.

6. Would this "expert" model be useful for the purposes stated if it were complete with all road warning, regulatory, and guide signs? (select a number)

	Little Usefulness							Great Usefulness	
Training	1( )	2( )	3( )	4( )	5( )	6( )	7( )	8( )	9( )	10( )
Evaluation	1( )	2( )	3( )	4( )	5( )	6( )	7( )	8( )	9( )	10( )

7. Should it include recreational, trail, Forest entrance, and administrative signs? Yes ( ) No ( )

8. Should we continue this effort, including changes as per your comments above? Yes ( ) No ( )

9. Would the system save you time? Yes ( ) No ( )

If yes, please estimate, with explanation, the amount of time that could be saved.

Thank You for Your Help!

General Comments (optional):

FIGURE 7 Questionnaire.

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The information contained in this paper reflects the views, opinions, and conclusions of the authors and does not necessarily represent those of the Forest Service.

Indian Low-Volume Roads: A Challenge for Maintenance Organization

M. P. PANDEY

Rural roads, which imply low-volume roads in India, have suffered a great deal for want of resources and organizational support. A paltry amount, without any rationale, is generally provided in the budget of the local authorities. The quality of work done is far below the standard specifications. In many cases, rural roads, bridges, and materials thereof become targets of vandalism at the hands of aggrieved persons and antisocial elements. The need for proper upkeep and maintenance of roads calls for augmentation of adequate resources and their proper utilization. Much of the problems come from inept organizational deficiencies largely emanating from centralized administrative control located in the state and district headquarters. In order to ensure the requisite flow of resources and their proper utilization, the involvement of local beneficiaries appears to be a great imperative. Their involvement will further facilitate preparation of an appropriate plan of road construction and its efficient execution. Beneficiaries' participation can be streamlined through the agency of Gram-Panchayat, a village level administrative unit consisting of a peoples' representative that already exists with fiscal, judiciary, and executive powers. The Gram-Panchayats are a symbol of decentralized administration. They will be ideal from the point of view both of efficiency and equity.

The economic development of a country and the advancement of civilization depend, among other things, on a good road system. In an economically underdeveloped country, there is an inadequacy of roads. The road network is deficient in length, unevenly dispersed, and poor in quality. Development of roads has been largely contingent on economic considerations but quite often on strategic grounds, too. Lately the growing trend toward internal insurgency, or persistent law-and-order problems, have also influenced the decision to construct new roads in many countries (*1*).

In India, low-volume roads comprise (a) feeder roads converging on national highways and state highways, (b) village access roads, (c) market link roads, (d) hill area criss-cross roads, (e) desert area criss-cross roads, (f) river bank roads, (g) canal bank roads, and (h) embankment or peripheral boundary roads. These roads, called "rural roads," have a common nomenclature in India. The rationale behind construction of these roads has flowed from a mix of factors.

Extensive recordings of road construction during the Mughal period are found in various published historical accounts (*2*). Shershah was credited to have constructed the Grand Trunk Road from Calcutta to Peshawar. Some other Kings also built trunk roads in other parts of the country. With the advent of railways, in the latter half of the 18th century, the trunk roads were neglected as most of the railways ran parallel to the then-existing trunk roads (*3*). However, for administrative convenience feeder and local roads proliferated. Some ideas for systematic road development emanated from two meetings of experts; one in the year 1943 at Nagpur, and the other in the year 1958 at Bombay. The outcome of the two meetings bears the popular nomenclature of the Nagpur and Bombay plans. Before that, another expert committee was appointed in 1927, called the Jayakar Committee, whose recommendations were not followed for various reasons. As per the Nagpur plan, roads were classified as (a) national highways, (b) provincial highways, and (c) district roads or village roads. It was decided that in a highly developed agricultural area the average distance of a village from a main road should be less than 2 mi. In a nonagricultural area, it was further decided that the average distance of a village from a main road should not be more than 5 to 6 mi. The Bombay plan did not mention unclassified village roads or municipal roads. The plan's broad objective was that no village in a highly developed area should be over 4 mi from a metaled or hard-surface road or over 1.5 mi from any type of road. In undeveloped and uncultivable areas, the targets were 12 and 5 mi, respectively, from (a) a metaled road; and (b) any type of road. In semideveloped areas, the corresponding distances were proposed to be kept at 8 and 3 mi (*4*), respectively. Figurative variation or improvement occurs between the Nagpur and Bombay plans. But that local decision making was completely absent in the development of the process is more important. Both plans were in the nature of a perspective plan, comprising indicative data, to which state government or local boards were expected to relate their efforts.

The next phase of road planning synchronizes with the five yearly plans, starting from 1950/51. The development of rural roads received little attention in the various 5-year plans. These plans largely concerned themselves with national highways, or trunk roads. In 1967, a special committee was set up to examine the status of rural roads and to suggest a suitable development policy. This committee perceived the crucial role of village roads in the context of speedy socioeconomic development of the countryside. It went a step further to declare that the villagers had an unquestionable democratic right to seek suitable roads from the state for quick and easy access to the outer world. The committee recommended certain planning criteria for rural roads, although the recommendations were not followed up (*5*). The Fifth Five Year Plan (1975/76 to 1979/80) envisaged a minimum needs program for the villages, which contained road needs as one of its segments. It kept the provision that all villages with a population of 1,000 or more should have an all-weather road, but this objective proved to be too big to be realized in the course of one 5-year plan. Later, the program was modified,

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according to which it was planned to link all villages with population of 1,500 or more with all-weather roads by the year 1990. In the Seventh Five Year Plan document (1985 to 1990), the importance of rural roads has been reiterated. It reads as follows:

> Since the country's economy is still largely agrarian in character and the settlement pattern is rural oriented, roads constitute a critical element of the transportation infrastructure. Road construction and maintenance generate sizeable employment opportunities, a factor that has assumed considerable importance with demographic expansion and the growth of the labour force. Better roads also help achieve fuel economy and improve the overall productivity of the Road Transport Sector (*6*).

The status of rural road network thus far achieved indicates that they connect 64 percent of the villages, though not with all-weather roads. About 18,000 villages were connected with roads under the Minimum Needs Program. A lot more needs to be done to improve the road system. About 36 percent of the villages in the country are still without any road connection, and as much as 65 percent are without an all-weather road. Rural roads are also constructed under other schemes such as the Rural Landless Employment Guarantee Program (RLEGP), National Rural Employment Program (NREP) and Command Area Development Program (CADP). A number of organizations handle road construction and maintenance resulting in duplication of effort, lack of uniformity, and the unbalanced development of the road network. There is a need to unify the organizational structure for rural road planning, construction, and maintenance so as to derive the maximum benefits from the relatively small outlays. While planning roads in hilly regions, consideration of the economy in costs and the preservation of ecological balance must receive due attention. While planning for roads in desert areas, alignment of roads should be closely related to serving water points. Roads should also be planned to link milk centers where processing plants are located.

The foregoing narrative throws light on the experience and vicissitudes faced by India in the course of planning and building rural roads. What strikes most vividly is the fact that the program of rural roads in the country has suffered a great deal for want of resources and organizational support. The effective expansion of the road network in the countryside requires adequate maintenance support to ensure full utilization by road users.

While reflecting on the problems of maintenance of rural roads in India, it is necessary to keep in mind the conditions surrounding road use in the country and the hazards to which the roads remain exposed for most of the time. Quite a lot of such roads are made of earth and mud. Such roads are used mostly by animal-driven carts having solid wood wheels with iron rings, tractors, and heavy trucks that damage the surface of the road during fair weather. The repeated operation, particularly of the solid-wood, iron-wheeled carts, causes deep cutting in the pavement (*7*). Such roads get eroded during rainy season. In flood-prone areas, very often even bitumen roads get washed away. Occasionally such roads get damaged by land slides or severe earthquakes.

Quite often, roads are cut deliberately to facilitate the flow of rain water in the cultivated fields on the other side of the roads. Roads sometimes become a shelter place for cattle and human beings in times of heavy flood. People come out of their flooded houses with all their belongings and stay on the roads till the water recedes. Roads also serve as a market place where one finds vendors, hawkers, and others selling fruits, vegetables, consumer goods, domestic materials, tea, snacks, and allied things. The settlers erect various kinds of structures to provide improvised shops, encroaching on a good portion of the roads. Such encroachments on the road block or obstruct drains. This results in overflowing of drain water, which then damages the crust or upper layers of the roads. They become unusable even by pedestrians.

Roads may also become a target for unscrupulous persons who, to serve their small needs, remove bricks, stone metals, girders of bridges, and other objects, in callous disregard of the interest of road users. Thus the roads, while serving the economy and society in varied ways, occasionally become a curse when they facilitate easy movement of thieves, bandits, terrorists, and similar antisocial elements, who make life miserable for the villagers. To the extent society suffers from such damages, necessary discount should be made in computing the value of the road network.

Maintenance of roads in the countryside of India has suffered chiefly from want of resources. A paltry amount without any rationale is generally provided in the budget of the local authorities or the department of rural development. This amount is hardly ever increased. It is nominal to begin with and it is even more insignificant if one takes into account the loss of its real worth in the wake of ever-rising inflationary prices. In absolute terms, the amount is minimal—and even that gets diverted to other purposes, if the government or the local authority is confronted with a problem of greater urgency (*8*). To cite examples from old records of Gaya District Board (Bihar State), provision for repairing "kuchcha" road in the jurisdiction of Aurangabad Local Board used to be a meager amount of Rs.15,000/- until 1948. The corresponding figure for Nawadah Local Board used to be even less, i.e., Rs.10,000/-.This fixed amount was not spent for even three consecutive years, because the amount used to be diverted for paying the salary of those who were called "work charged staff." The situation was even worse in the case of funds provided for repair of kuchcha roads in the jurisdiction lying with Hajipur and Sitamarhi Local Boards within Muzaffarpur District Board. Whenever some dubious expenditure used to be incurred by the authorities of such District/Local Boards, the amount so spent used to be falsely charged to the head of repairs of kuchcha roads. Often, the earthwork for repairing kuchcha roads was undertaken just about the time when the monsoon was about to set in. The purpose behind the timing of this repair was to camouflage the measurement of the earth cutting. Invariably, an inflated quantity of earth cut used to be shown in order to charge higher prices for the work done. The quality of work done was far below the specifications prescribed in the works manual of the State Works Department (*9*). Pilferage of road materials, such as stone chips, bricks, cement, bridge girders, hume pipes, iron poles, beams, etc. is reported to be rampant in all parts of Bihar state. Such pilferage takes place not merely from work sites when repair or construction is going on, but even from places where construction is complete and roads, bridges, and culverts have been in use for a long time. Although such nefarious acts are indulged in by unscrupulous persons mostly for

petty personal gains, often they occur in the form of vandalism, an expression of some kind of revengeful action against the government, or against the people of certain areas. People give vent to their suppressed anger or acrimony against the government by indulging in acts of vandalism on government property, such as roads, transport vehicles, railways, etc. In India, many militant organizations have risen to serve certain political ends, such as reservation in government services for backward castes, scheduled castes or tribes; regional autonomy; change in language policy; price supports for agricultural commodities; elections to regional or local authorities, etc. The roads, bridges, and culverts become easy prey to these group actions. Such acts add a further dimension to the question of maintenance and upkeep of rural roads, which lie obscurely hidden from the view of law and order authorities. In any case, it is difficult to ensure proper safety to roads, bridges, etc., which are located beyond the limits of towns or villages.

The problem of maintenance of roads, especially village roads is to be viewed in this perspective. What are the matters of most concern if one has to suggest some policy measures for providing fool-proof maintenance and upkeep of rural roads? Such policy measures should be considered in keeping with conditions found in resource-poor developing countries. Looking closely at the problem, cost becomes the first object of concern. Resources in rural areas in the developing countries are always meager, as have been indicated. Their augmentation is a major problem. To put them to the best use is another problem. The twin problems of resources augmentation and their optimum allocation can be better ensured by changing the organizational support. Hitherto, the work used to be done by a department of government or local bodies. In India, the countryside is widely dispersed and limited funds get spread out too thinly to be of much use. Besides, a central organization cannot plan or execute the job properly. To assign the work to each village community would be better.

At the village level, there is a body called the "Panchayat" that functions as a development council and also as an administrative unit. It has the statutory power to raise resources, to plan development programs, and to execute them. This body promotes law and order and is vested with the legal power to punish law breakers. The Panchayats have thus multiprong functions in revenue, administrative, and judicial spheres. This body is ideally suited to plan, execute, monitor, supervise, and oversee the entire process of work involved in construction and maintenance of rural roads. These bodies also have the power to involve the surplus labor power of the rural sector in maintenance works. They can utilize local resources in cash or kind from the people on a pro-rata basis. They can remove unauthorized encroachments or structures from the roads. They can ensure the protection of removable parts of roads and bridges by antisocial elements. They can prevent unauthorized road cuttings by individuals to divert water from one side of the road to the other for personal convenience. They can keep guard against drain blockers and thereby prevent drain water from flowing on the roads. This body also has the power to requisition the services of experts and consultants from the central pool to answer technical matters or intricate problems related to road construction and maintenance. The involvement of the Panchayats in road maintenance would be ideal from an efficiency point of view as well as an equity point of view. Already, we have such organizations working in different parts of India to generate and distribute irrigation water. In the state of Maharashtra there are cooperatives doing a similar job for the benefit of cane growers. In Gujrat, the Anand dairy is a massive organization devoted to the job of promoting the socioeconomic status of small milk producers. The irrigation associations of Taiwan, Indonesia, and Thailand are doing a similar job. Panchayats that are vested with legal and constitutional powers are better suited to take care of the roads in the rural areas, and these bodies have the advantage of being able to involve the community on a voluntary basis. They also have the teeth to enforce rules and guidance that the community formulates to promote their collective interest. This experiment should be undertaken on a high-priority basis in all developing countries.

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Review of Surfacings for Low-Volume Roads on the Basis of Experience in South Africa

H. WOLFF AND A. T. VISSER

Local and overseas specifications and design methods of single, double, slurry, Cape, sand, primer, and Otta seals were compared to investigate potential surfacings for low-volume roads. The following conclusions were drawn: (a) aggregate is rigorously specified for single, double, slurry and Cape seals and little or no relaxation has been considered, whereas the specifications for sand, primer and Otta seals are not as rigorous; (b) binder is rigorously specified by relevant South African Bureau of Standards specifications for all seal types; and (c) formal design methods exist for single and double seals, but the designs of the other seal types are mostly empirical. A calculation of life cycle costs to road authorities over a 20-year period indicated that single and double seals are the most cost-effective over a wide range of traffic volumes. Low unit construction prices do not necessarily mean high cost-effectiveness, as the expected lifetime of the seal is an important consideration.

In many developing regions, traffic volumes are low and consequently local natural materials are usually used for the pavement layers. This process allows cost-effective provision of the road infrastructure, and also helps to stimulate local development. There is, however, one costly item in providing paved roads, namely the surfacing. Surfacing standards appropriate for higher traffic volumes have typically been used in the low-volume environment. This choice has frequently precluded the use of local materials, which may not comply with the rigid specifications required by the surfacing standards. The aim is to critically review the factors that affect the performance of potential surfacing types and, on the basis of experience in South Africa, to present guidelines for the use of these surfacings for low-volume roads. Although many roads in South Africa could be categorized as carrying medium-to-heavy traffic, more than 75 percent of the rural roads are surfaced with chip seals. The performance of surfacings is influenced by the characteristics and quantities of aggregate and binder used for constructing the surfacing, the support beneath the surfacing, and the traffic volume.

SPECIFICATIONS AND DESIGNS OF SEALS

Single and double seal

Aggregate is usually specified by grading, flakiness index, crushing strength (ACV or 10 percent FACT) and cleanliness or dust content. A typical specification (*1*) is presented in Table 1 for stone with a nominal size of 13.2 mm. This specification calls for surfacing stone of a fairly high standard. The strength requirement is the only characteristic that has been relaxed to as low as 80 kN (10 percent FACT) for low-volume roads (*2,3*). No relaxation of specifications for grading, flakiness index, cleanliness or dust content of the surfacing stone could be found in the literature (*4–8*). There are, however, different views on whether particle size and form of the stone should be relaxed at all (*2,9*).

Binder is specified by relevant standards, e.g., South African Bureau of Standards (SABS) specifications, which are often based on ASTM standards. Manufacturers have to comply with these specifications and no relaxation has been considered.

Design methods differ between road authorities. A rational design method was, however, developed by the Division for Road and Transport Technology, which lends itself to the design of surfacings for low-volume roads (*10–13*). With this method, the amount of binder required in a single or double seal is determined as a function of the void volume of the aggregate in the seal. A certain percentage of these voids must be filled with binder. The void volume on the other hand is dependent on the stone size, the embedment into the underlying layer, and the wear of the stone under traffic. Embedment can be determined with the ball penetration test (*14*). Relationships between traffic and embedment exists for traffic volumes as low as 125 vehicles per day (vpd) and bases as soft as 8 mm of ball penetration. The wear of the stone under traffic is a function of the crushing strength of the stone. Relationships between the 10 percent FACT value and wear exist for traffic volumes as low as 125 vpd and 10 percent FACT values as low as 150 kN. If these relationships can be extended for traffic volumes as low as, say, 50 vpd, soft bases (existing gravel road surfaces) and surfacing stone with low 10 percent FACT values, then the rational method developed by the Division for Road and Transport Technology can be used to design seals for low-volume roads with soft bases and aggregates with low crushing strengths.

Slurry Seal

Aggregate specifications for slurry seals are given in Table 2 (*15*). The Committee of State Road Authorities (CSRA) specification is used most frequently in South Africa. Aggregate quality is rigorously specified since it has been found that

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TABLE 1 TYPICAL SPECIFICATION FOR 13.2-mm NOMINAL-SIZE SURFACING STONE (*1*)

GRADING		CRUSHING STRENGTH	FLAKINESS
Sieve Size (mm)	Percentage passing by mass	Maximum aggregate crushing value (%)	Maximum flakiness index (%)
19,0	100	21	25
13,2	85-100		
9,5	0-30		
6,7	0-5		

relaxation of the specifications generally led to poor performance (*16*). Specifications permit a sand equivalent (*17*) as low as 35, but experience has shown that the sand equivalent should preferably be not less than 45.

The bitumen emulsion used for slurry manufacture is predominantly of the stable-grade anionic type as specified in SABS 309 (*18*). A variety of bitumens from 60/70 to 150/200 penetration can be used as the base bitumen in emulsion for slurry sealing. However, the bitumen selected is usually between 80/100 and 150/200 penetration, the lower penetration bitumens being used in the hot areas and for heavy traffic conditions, and vice versa (*19*). It is estimated that about 95 percent of emulsions used in South Africa for slurry sealing are of the anionic type and are used mainly with dolerite, dolomite, quartzite, and norite aggregates. This is in contrast to the rest of the world in which cationic emulsions are favored (*15*). The Transvaal provincial administration, for example, uses only anionic emulsion because cationic slurries tend to break too soon.

Cationic bitumen emulsion as specified in SABS 548 (*20*) has also been used in South Africa, mainly with quartzites and natural sands. Cationic emulsions give less stripping and consequently greater durability than anionic emulsions with aggregates that are hydrophilic (*15*).

Design methods are mostly empirical or semiempirical (*15*). Use of these simple design methods (aggregate gradings and bitumen content specified) is appropriate when good-quality aggregate can be obtained. When the use of aggregate that is outside the specification is considered for low-volume roads, the more sophisticated design methods such as the wet track abrasion test (WTAT) (*21–24*) and cohesion test (*25*) are preferred.

Cape Seal

Cape seal is a surface treatment consisting of a 9.5-, 13.2-, or 19-mm layer of stone placed shoulder to shoulder on a tack coat and covered with one or two applications of slurry so that the tops of the stones are left proud (*26,27*).

Aggregate specifications for Cape seals are essentially those for single and slurry seals. Aggregate of a high standard is thus specified and no examples of Cape seals built from lower standard material could be found.

Binder is usually one of, or a combination of, the following (*27*):

- Road tar (RTH 50/55), bottom spray of 19-mm surfacings;
- 150/200 penetration bitumen, bottom or cover spray;
- MC3000 cutback bitumen, bottom or cover spray;
- Cationic spray-grade emulsion, bottom or cover spray;
- Cationic stable-grade emulsion, slurry manufacture; or
- Anionic stable-grade emulsion, slurry manufacture.

Road tar is used in areas where it is cheaper than bitumen, but then only for the bottom spray of 19-mm surfacings with the cover spray consisting of a bitumen product. These binders are also specified by relevant SABS specifications and no relaxation has been considered.

Design of Cape seals is mostly empirical (*1,26,27*). The stone sizes for the single seal are related to traffic volume. The slurry is designed by specifying aggregate properties for a specific bitumen content. Cape seals have never been designed for low-volume road applications. Design methods proposed by Biesenbach and Alexander (*27*) allow for two categories of traffic: less than 1,000 vehicles per day (vpd) and more than 1,000 vpd. Although Campbell (*26*) provides for as low as 200 vpd in his design methods, the minimum traffic volume that Cape seals are normally used for is about 500 vpd (*27*). Aspects mentioned under single and slurry seal design should be considered when designing Cape seals for low-volume roads.

Sand Seal

Sand seals can be described as consisting of a cutback bitumen or emulsion tack coat and a suitably graded sand applied to a base course with or without a prime coat (*28*).

Aggregate gradings for sand seals are given in Table 3, which indicates that the TRH14 (*4*) and Mitchell (*28*) gradings are almost identical. These two gradings fall between the ideal and average grading proposed by Marais (*5*). Grading is the only aggregate property specified in the literature studied. The gradings proposed in Table 3 fall within a wide envelope; therefore, the aggregates that can be used in sand seals are not rigorously specified. However, some sand seals last longer than others. This can probably be attributed to the aggregate used for the construction of the seal. Further investigation

TABLE 2 STANDARD SOUTH AFRICAN SPECIFICATIONS FOR SLURRY AGGREGATES (*15*)

Requirement	SABS 1083	CSRA*			TRH3&TRH7 NITRR	CPA+	TPA+		NPA+	OTHER(21)		
		Coarse	Medium	Fine			Coarse	Fine		General Medium	Normal Coarse	Extra Coarse
Grading												
9,5mm	100	100	100	100	100	100	100	100	100	100	100	85-100
6,7mm	100	100	100	100	100	100	100	100	100	100	85-100	70-90
4,75mm	100	70-90	82-100	100	100	100	70-90	100	100	85-100	70-90	60-83
2,36mm	90-100	45-70	56-95	90-100	90-100	90-100	45-70	90-100	90-100	65-85	45-70	40-60
1,18mm	65-95	28-50	37-75	65-95	65-95	65-95	28-50	65-95	65-95	45-70	28-50	28-45
0,600mm	42-72	19-34	22-50	42-72	42-72	42-72	19-34	42-72	42-72	30-50	19-34	19-34
0,300mm	23-48	12-25	15-37	23-48	23-48	23-48	12-25	23-48	23-48	18-30	12-25	14-25
0,150mm	10-27	7-18	7-20	10-27	10-27	10-27	7-18	10-27	10-27	10-21	7-18	8-17
0,075mm	4-12	2-8	4-12	5-12	5-12	3-12	5-15	4-12	4-12	5-15	5-15	4-8
Maximum addition of suitable sand	25%	25%			25%	25%	25%		50%	N.S		
Fineness modulus**	2,0-3,8	N.S.			N.S.	N.S.	2,0-3,8		2,0-3,8	N.S.		
Minimum sand equivalent	35	35			N.S.	N.S.	35		35	N.S.		
Maximum ACV (Parent rock)	N.S.	30			N.S.	N.S.	30		30	N.S.		

N.S. - not specified

* Committee of State Road Authorities.

** Sum of cumulative percentages retained on the 4,75; 2,36; 1,18; 0,600; 0,300 and 0,150 mm sieves divided by 100.

\+ CPA; TPA & NPA - Cape; Transvaal & Natal Provincial Administrations

TABLE 3 PROPOSED GRADING FOR SAND TO BE USED IN SAND SEALS (*4,5,28*)

Sieve Size (mm)	PERCENTAGE PASSING BY MASS				
	MARAIS			TRH 14	MITCHELL
	Ideal	Average	Fine		
9,5	100	100	100	-	-
6,7	-	-	-	100	100
4,75	85-100	85-100	100	-	-
2,36	-	-	85-100	-	-
1,18	25-50	50-85	-	-	40-65
0,600	0-20	20-50	50-85	-	10-35
0,300	0-5	0-10	-	0-15	0-15
0,150	-	-	-	0-2	0-2
0,075	0-2	0-2	0-2	-	-

into the relation between the properties of the aggregate and the performance of the seal is thus warranted. This would enable designers to relate the expected performance of the sand seal to the quality of the aggregate used. Although sand seals were popular about 10 years ago in South Africa, variable performance and the difficulty in obtaining suitable sands has led to a preference for small-size single seals.

Binders used for sand seals are again rigorously specified by the relevant SABS specifications and no relaxation has been considered.

No formal method of design exists. Application rates for tack coat and aggregate proposed by two researchers are presented in Table 4 (*28,29*).

The proposed binder application rates do not correlate well. It is assumed that both rates are actual hot spray rates. Performance is most likely also subject to the amount of binder in the seal, although this has not been fully investigated. Further investigation into this aspect could lead to different application rates for different traffic volumes and consequently more cost-effective designs for low-volume roads.

Primer Seal

Primer seal is a combination of a priming and sealing operation and consists of the application of a primer binder with a viscosity midway between that of a primer and a seal bitumen covered by a layer of aggregate. This can be a single-size aggregate of nominal size 5, 7, or 10 mm or a graded crushed-rock aggregate or natural sand that mostly passes the 4.75-mm sieve (*6*).

Aggregate gradations for primer seals are presented in Table 5 (*5,30*). A comparison of the grading proposed by Marais for the sand seal and the primer seal reveals that the grading for the primer seal is much coarser than for the sand seal. However, the National Association of Australian State Road Authorities (NAASRA) primer seal grading (*30*) appears to correspond with the Marais sand seal grading (*5*), whereas the grading for the primer seal proposed by Marais is much coarser than even the ideal primer seal grading proposed by NAASRA.

There may be a small difference between the grading proposed for sand and primer seals, and Marais, in fact, uses the NAASRA primer seal grading for a sand seal. However, the primer seal grading proposed by Marais is considerably coarser than the grading proposed by NAASRA. As far as the aggregate is concerned, sand seals and primer seals are identical. Judging by the wide range of proposed gradings, the grading for a sand or primer seal may not be critical. It is also the only aggregate property specified. This then is the ideal type of surfacing for low-volume roads, as a wide range of materials can be used. However, the grading and strength of the aggre-

TABLE 4 TACK COAT AND AGGREGATE APPLICATION RATES FOR SAND SEALS (*28,29*)

	VISSER(28)	MITCHELL(27)
Tack coat (l/m^2)	1,25	1,5 - 2,0
Aggregate (m^2/m^3)	-	200

TABLE 5 AGGREGATE GRADING PROPOSED FOR PRIMER SEALS (*5,30*)

Sieve size (mm)	Percentage passing by mass				
	NAASRA			MARAIS	
	Ideal	Average	Fine	10 mm	7 mm
13,2	-	-	-	100	100
9,5	100	100	-	85-100	100
6,7	-	-	-	0-40	85-100
4,75	85-100	85-100	100	0-10	-
3,35	-	-	-	-	0-30
2,36	-	-	85-100	0-1	0-10
1,18	25-50	50-85	-	-	-
0,600	0-20	20-50	50-80	-	0-2
0,300	0-5	0-10	-	-	-
0,075	0-2	0-2	0-2		

gate must in some way influence the performance of the surfacing, and this aspect should be investigated further.

Binders for primers seals are again specified by the SABS and no relaxation has been considered.

Binder and aggregate application rates are presented in Table 6 (*5,6*). These rates correlate well with the application rates for the sand seal given in Table 4. The application rates proposed by Marais (*5*) allow for traffic volumes of less than 250 vpd and higher than 2,000 vpd, but do not accommodate the intermediate traffic volumes. The rates proposed by Dickinson (*6*) agree with the rates proposed by Marais for the RTH/RTL 35/40 and MC800 binder types. These binder types require less binder for the same aggregate application rate than the RTH/RTL 40/45 and MC3000 types. The binder application rates are well defined by Marais and Dickinson and correlate well. Because of the similarity in aggregate grading between the two seal types, the rates proposed for primer seals could probably be applied to sand seals as well.

TABLE 6 BINDER AND AGGREGATE APPLICATION RATES FOR PRIMER SEALS (*5,6*)

	Binder Type	Traffic Volume (vpd)	Application rates Binder (l/m^2)	Aggregate (m^2/m^3)
MARAIS (5)	RTH/RTL 35/40	<250	1,35	170-200
	MC800	>2000	1,10	170-200
	RTH/RTL 40/45	<250	1,50	125
	MC3000	>2000	1,20	125
DICKINSON (6)	-	<250	1,30	100-200
	-	250-2000	1,20	100-200
	-	>2000	1,10	100-200

Otta Seal

Otta seal is a surface dressing method using traditional equipment, working techniques, and binder. However, the covering aggregates are quite different from those conventionally used.

Although the traditional surface dressing requires more or less single-sized, dust-free stone, the Otta surfacing method prescribes the use of naturally occurring sandy gravel, including fines. The maximum size of the gravel can vary from 12 to 20 mm. The gravel is used at its natural water content, which usually varies from 3 to 5 percent by mass. Compaction is carried out by the combined effect of rolling and the addition of an adhesion-active agent (fatty amine) to the binder. The subsequent traffic also contributes substantially to the compaction.

Aggregate grading for Otta seals proposed by Marais (*5*) and Thurmann-Moe and Ruistuen (*31*) is presented in Table 7. The material is more uniformly graded and much coarser than aggregate used for primer and sand seals. The gradings proposed by the two researchers are similar. The grading requirements for the covering aggregate are modest and are normally the same as for base course material, although the maximum size is smaller.

Binder and aggregate spray rates proposed by Thurmann-Moe (*31*) and Marais (*5*) are presented in Table 8. No distinction is made between different traffic volumes in the design procedure.

As natural aggregates are used, the Otta seal has great potential as a surfacing option for low-volume roads. However, binder application rates for low traffic volumes will have to be established by further investigation.

COST-EFFECTIVENESS OF VARIOUS SURFACING TYPES

Cost-effectiveness should take cognizance of the costs to road authorities as well as to the road user. Costs to the authorities comprise initial construction as well as maintenance costs. User costs comprise vehicle operating, time, and collision

TABLE 7 GRADING OF NATURAL AGGREGATES FOR OTTA SEAL AS PROPOSED BY MARAIS (*5*) AND THURMANN-MOE (*31*)

Sieve Size (mm)	PERCENTAGE PASSING BY MASS		
	MARAIS		THURMANN-MOE
	Nominal 0-12 mm	Nominal 0-20 mm	
19,0	100	100	100
13,2	100	60-100	50 - 100
9,5	70-100	40-70	31 - 90
4,75	40-70	15-40	18 - 68
2,36	30-50	8-30	10 - 52
1,18	20-40	5-20	5 - 40
0,600	15-30	3-15	3 - 32
0,300	12-22	2-12	2 - 20
0,150	8-15	1-8	0 - 15
0,075	5-10	1-5	0 - 10

TABLE 8 BINDER AND AGGREGATE SPRAY RATES FOR OTTA SEALS RECOMMENDED BY THURMANN-MOE (*31*) AND MARAIS (*5*)

Nominal Size (mm)	Aggregate Spread Rate (l/m^2)	Binder Spray Rate (l/m^2)
0-12	12-15	1,5
0-20	20-25	2,0

costs. However, user costs are not considered in this analysis, which is assumed to be the same for all surfacing types. This assumption will, however, not be valid when surfaced roads are compared to gravel roads. The cost of resealing is considered to be the maintenance cost of the surfacing over the analysis period.

The life of a surfacing is a function of the following:

- Type of surfacing;
- Volume and type of traffic;
- Support from pavement structure;
- Quality of surfacing construction; and
- Environmental conditions, e.g., temperature and rainfall.

Table 9 presents estimates of the expected life of the surfacing types under the traffic volume ranges for which the surfacings are normally used. These data (except for the Otta seal) were collected by the Transvaal provincial administration over a number of years and assume a good-to-reasonable pavement condition. They represent about 70 percent of all the cases encountered (G. D. van Zyl, unpublished data). The factors influencing surfacing life must also be considered when a life cycle cost analysis is conducted using the information presented in Table 9.

Table 10, which presents the life cycle costs to the authorities over a period of 20 years [defined as the present worth of cost (PWOC)], was compiled from the unit costs presented in Table 10 and the expected lifespans and traffic volumes presented in Table 9.

A discount rate of 8 percent was used. In those cases for which 20 years was not a multiple of the lifespan of the seal, the salvage value at 20 years of the last reseal was taken as the unit price multiplied by the proportion of the surfacing life that still remains.

Table 10 can be used as a guideline for the type of seal to be selected for specific situations. However, the factors influencing surfacing life mentioned earlier must be kept in mind when a selection is made. This table indicates that single- and double-chip seals are cost-effective in most situations. Low unit prices do not necessarily mean high cost-effectiveness, because the expected lifespan of the seal has to be taken into account. Although this table is applicable to South African conditions, the logic used in developing the table as well as the surfacing lives can guide practitioners in other environments.

CONCLUSIONS

In South Africa, a proud and successful tradition in the use of surfacing seals has been established on low-to-medium-heavily trafficked roads. This approach could serve as a model for low-volume roads in a similar environment. In this review, the potential surfacing types were evaluated and the following conclusions regarding specifications, design methods, and use of surfacings for low-volume roads can be made:

- Binder to be used for the construction of the surfacing types is well specified from long-term experience, and no

TABLE 9 EXPECTED LIFESPANS AND ASSOCIATED TRAFFIC VOLUMES OF CERTAIN SURFACINGS (*31* AND G. D. VAN ZYL, UNPUBLISHED DATA)

SEAL	LIFESPAN (years)	TRAFFIC VOLUME (corrected vpd)*	TRAFFIC CATEGORY
Sand	3-5	400-2 000	1
Slurry (fine)	3-5	2 000-4 000	1-2
Single (9,5 mm)	10-12	2 000-4 000	1-2
Double Otta	10	1000 ADT**	1-2
Single (13,2 mm)	10-14	4 000-20 000	1-3
Slurry (coarse)	4-8	4 000-20 000	1-3
Cape (13,2 mm & slurry)	8-12	4 000-20 000	1-3
Double (13,2 mm & 6,7 mm)	10-14	4 000-20 000	1-3

* vpd: vehicles per day
corrected vpd: 1 heavy vehicle = 20 vehicle units

** ADT: Average Daily Traffic - assumed to be vehicles only with no correction for heavy vehicles. If 15 per cent heavies are further assumed, 1000 ADT will be category 2 traffic.

TABLE 10 LIFE CYCLE COST TO AUTHORITIES OVER 20 YEARS (PWOC, DISCOUNTED AT 8 PERCENT) FOR VARIOUS SURFACING TYPES

SEAL	LIFESPAN (years)	UNIT PRICE (R/m^2)	PWOC (8%) (R/m^2)		
			TRAFFIC CATEGORY		
			1	2	3
Sand/primer	4	1,55	4,59*	-	-
Slurry (fine)	4	1,61	4,77	4,77	-
Single (9,5 mm)	10	1,75	2,56	2,56	-
Otta (double)	10	3,30	4,83	4,83	-
Slurry (coarse)	6	3,26	6,96	6,96	6,96
Cape (13 mm & slurry)	10	3,69	5,40	5,40	5,40
Double (13,2 mm & 6,7 mm	12	3,38	4,48	4,48	4,48

* Example: $PWOC = 1{,}55\{1 + 1/(1{,}08)^4 + 1/(1{,}08)^8 + 1/(1{,}08)^{12} + 1/(1{,}08)^{16}\}$

$= 4{,}59$

relaxation is considered possible without jeopardizing performance.

- Aggregates for single and double seals are well specified and carefully developed design methods exist. However, the specifications and rational design method could be extended to cater for lower traffic volumes.
- Aggregates for sand, primer, and Otta seals are generally not well specified, which may account for the considerable differences in performance of these surfacing types.
- Binder application rates for sand, primer, and Otta seals are also not well defined. This fact may also account for differences in performance.
- Single- and double-chip seals are cost-effective in most situations, as was demonstrated by the South African experience.
- The expected lifespan of the seal is an important consideration when determining cost-effectiveness.

RECOMMENDATIONS

The effect of aggregate properties and binder application rates on the performance of sand, primer, and Otta seals should be further evaluated to use these surfacing types for low-volume roads most cost-effectively.

The relationship between traffic and embedment of aggregate into road bases used in the rational design method of the Division for Road and Transport Technology should be extended to allow for traffic volumes lower than 125 vpd and bases softer than 8 mm of ball penetration. Furthermore, the relationship between the wear of stone under traffic and the crushing strength of the stone should be extended to allow for traffic volumes lower than 125 vpd and for aggregate with 10 percent FACT values of less than 150 kN. This condition will allow the rational design method to be used cost-effectively in the design of single and double seals for low-volume roads.

The following additional factors for defining appropriate seals for a specific project should be considered:

- In cases of no or irregular maintenance, seals which do not need regular maintenance, such as Cape seal, should be used.
- Because no general seal that would be most economical for all situations exists, each situation should be evaluated individually.
- Aggregate properties and binder application rates should relate to performance required.

ACKNOWLEDGMENTS

This paper is published with the permission of the South African Bitumen and Tar Association (SABITA) and the Director of the Division of Roads and Transport Technology, CSIR. The sponsorship by SABITA for the research work and their

approval for the publication of this paper is greatly appreciated. The support from colleagues at the CSIR is also acknowledged with thanks.

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Selection and Designation of Scenic Byways: A Quantitative Approach

BOB L. SMITH AND WILLIAM L. SMITH

In the selection and designation of scenic byways, there is a need for quantitative criteria to ensure minimum acceptable levels of scenic and historic quality. A study approach to the quantification of byway quality in the states of Iowa, Kansas, Missouri, and Nebraska is described. A system consisting of a lap-top computer and a video camera connected to a distance measuring device (DMD) is used on board a vehicle to collect information about a potential byway. A commentator (usually the driver) describes the following: the type of view (panorama, scene, or focal point); the quality of view with a numerical rating from 1 (excellent) to 5 (poor or highly detracting); and the quality of presentation based on the relative ease of seeing the various views as the road is driven. The views are given a quality of presentation rating from 1 (straight ahead) to 5 (out the side window); how long (distance) a particular view or element is observable; the types of activities along the road; and a 1 to 5 rating of the visual character of the roadway itself. The information from the commentator is stored in the computer using a specially coded and colored keyboard. Certain keystrokes poll the DMD to collect distance speed and time. The video camera is panned to record the view being described by the commentator. It captures the verbal comments as well as the instantaneous distance, speed, and time. The quantitative approach will be used to determine potential byway rating numbers for use in the selection and designation of scenic byways.

A scenic road or byway has roadsides or corridors of aesthetic, cultural, or historic values. The economic value of scenic roads to a state's economy is underestimated by most persons. An essential part of this road is its scenic corridor. The corridor may contain outstanding scenic vistas, unusual geologic formations, dramatic urban scenes, scientific features, or other elements—all providing enjoyment for the highway traveler (*1*).

There is now a nationwide effort (the Scenic Byways Movement) to focus attention on the need to develop scenic byways and on their potential for enhancing tourism and recreation; to create coalitions and strategies to actually develop the byways; and, alternately, to see that the job gets done.

There is also a great deal of interest in establishing or designating scenic byways in Iowa, Kansas, Missouri, and Nebraska. The transportation departments of each of the four states and the Midwest Transportation Center, the U.S. Department of Transportation–funded research center for the four-state region, operated by Iowa State University and the University of Iowa, have sponsored a scenic byways research project at Kansas State University. The research project, "Scenic Byways: Their Economic Benefits/Selection/Designation/Protection and Safety" (The Byways Project) was started in August 1989, and the engineering segment, i.e., all except the economic benefits portion, is scheduled for completion September 30, 1990. The principal investigator for the engineering segment of the project is Bob L. Smith.

A portion of engineering segment of the Byways Project is described in the following sections.

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BACKGROUND

In order to achieve consistency in the selection of future designated Scenic Byways, one must be able to promise some minimum level of scenic and historic quality. It is generally believed that many groups will want their road to be one of the designated scenic byways primarily because of the perceived economic benefits of byway designation.

Some organization such as a state or local road agency or state byway committee must be able to accept or reject the request for scenic byway designation for a given road. Vermont (*2*), New York (*3*), Arizona (*4*), and Colorado have well-defined procedures for scenic and historic byway designation.

The organizations responsible for designating scenic and historic byways need quantitative criteria to assure minimum acceptable levels of scenic or scenic and historic quality. Vermont (*2*), New York (*3*) and Arizona (*4*) have guidelines that are somewhat quantitative but are principally qualitative. In the evaluation of a potential scenic road in Vermont and New York, for example, the evaluators are to check positive values for scenic road criteria such as vegetation, landscape features, road characteristics, waters, buildings, other man-made structures, and others. Each of these criteria has several subcategories, e.g., under landscape features there are such items as panoramic and other distant views; scenic terrain (near and middle distance); natural focal points; and cliffs, boulders, and rock outcrops.

There is also a list of negative criteria such as landscape scars; buildings; other man-made structures; and the category, other. Each negative criterion has several specific subcategories, as for the positive criteria.

Checks are made on a checksheet for each 1/10 mi for the various positive or negative values. It is recommended that a minimum net score of 10 for each mile is generally needed to qualify that road as a scenic byway.

STUDY APPROACH

Many of the basic concepts from New York (*3*) and Vermont (*2*) were used in developing the study approach to the quantification of byway quality.

The following items are important in rating potential scenic byways:

- What one sees;
- What one does;
- What one learns;
- The quality of what one sees;
- How long (distance) one sees a particular view or element;
- How the view is presented or displayed, i.e., straight ahead or out the side window;
- Types of activities along the road;
- Visual character of the road itself, i.e., how well it fits the terrain, how smooth and flowing is the "ribbon-of-roadway"; and
- Measures of variety or lack of variety (degree of monotony).

In order to relate these items to "what we can see and do when driving the roads," two basic rating systems were established. The first system deals with the evaluations of the visual road or corridor. The second system deals with the identification of the social road or corridor. The social aspects of the roadway deal with the cultural, historic, educational, and recreational activities that exist along the road corridor.

Visual Road Corridor

The visual road corridor was divided into the following elements for evaluation:

Type of View

1. Panoramas

A panorama is a sweeping, distant view that fills a wide arc of the road traveler's vision. Such views normally occur at high spots in the road where trees and brush have been cut to provide a wide overlook. For panoramas, there were 13 items related to materials and color.

2. Scenes

A scene can be any one of a wide variety of views which are generally closer than a panorama. A scene usually covers a much smaller portion of the traveler's vision. For scenes, there were 17 items related to materials and color.

3. Focal Points

Focal points are those views that are so framed that one's eyes are inevitably drawn to them. A focal point covers only a few degrees of the field of vision—a lone tree or building are common focal points. For focal points, there were 16 items related to materials and color.

Quality of View

The following quality of view ratings (1 to 5) were used for each type of view.

- 1 excellent
- 2 good
- 3 average (typically a 3 rating is not identified in driver commentary)
- 4 less than desirable—detracts, distracts
- 5 poor—highly detracting

Note that 1 and 2 ratings are the equivalent of positives and 4 and 5 ratings are the equivalent of negatives; e.g., 1 = positive 2, 2 = positive 1, 4 = negative 1, and 5 = negative 2.

Quality of Presentation

A quality of presentation of display of view rating 1 to 5 was used for each type of view. The quality of presentation is based on the relative ease of seeing the various views as the road is driven. As shown in Figure 1, those views that are straight ahead are easiest to see and are therefore given a score of 1.

There are several conditions that can result in a presentation score of 1. The obvious condition is near view of the ribbon of roadway that is always straight ahead. A straight road that

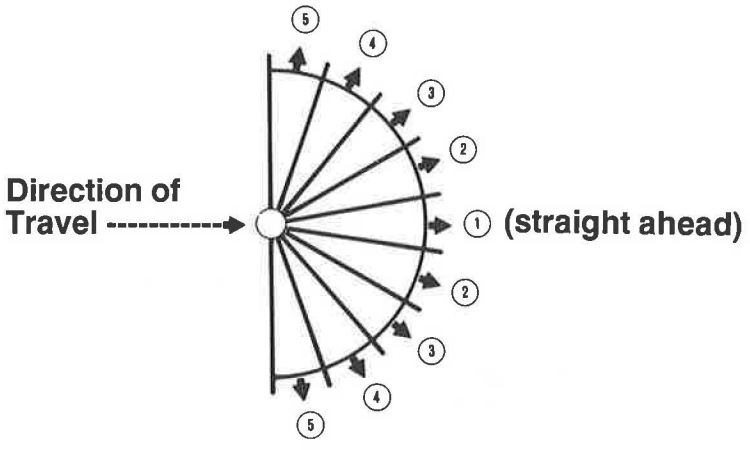


FIGURE 1 Quality of presentation rating scheme.

rises to a crest and allows the driver to overlook a valley straight ahead as the road falls and turns away would provide a presentation score of 1 for the view of the valley. Curving roads offer the most opportunities for presentation ratings of 1. As the road curves, the straight-ahead views coincide with the tangents to the curve as the driver moves along the curve. These tangent or straight-ahead views, as shown in Figure 2, are given presentation ratings of 1.

Those views that can be seen only by looking out the side window, the most difficult to see, are given a 5.

Distance a Given View is Visible

This parameter is measured automatically by the system used in the study.

Monotony Versus Variety

This parameter is likely to be measured by rate of view changing.

The Social Road Corridor

The social road corridor was identified by the type and number of the amenities within the corridor and the visual character of the roadway alignment. The social road or corridor was divided into following elements for evaluation:

- Visual and functional character of the roadway alignment.
  - —Quality of roadway appearance or presentation (rating of 1 to 5 is provided for evaluation),
  - —Roadway character resulting from good coordination of highway geometry was reflected through the ribbon of roadway element,
  - —Traffic, and
  - —Intersections and roadside activities.
- Educational and cultural elements that exist along the roadway corridor or can be accessed from the corridor, including historic structures, examples of ethnic activities, or museums.
- Natural features, including wildlife, geology, and vegetation.
- Amenities along the roadway such as motels, parks, or rest areas.

SYSTEM FOR GATHERING FIELD INFORMATION

The physical system, carried onboard the driven vehicle, consists of

- A distance measuring device (DMD) that is connected to the vehicle transmission and is ported for the lap-top computer.
- A lap-top computer with a specially coded and colored keyboard for inputting verbal comments. Certain keystrokes poll the DMD to collect distance, speed, and time.
- A software package for input/output and analysis of collected byway information.
- A video camera that captures the verbal comments of the commentator as well as desired roadway views. It also displays the instantaneous distance, speed, and time.
- A video cassette recorder (VCR) unit.

The physical system is an adaptation of the Route Inventory Information Management System (RIIMS) developed by Decision Data, Inc.

The system operating team consists of

- A commentator (usually the driver) who comments on the panoramas, scenes, or focal points with materials and color; the scenic quality (1 to 5); and the quality of presentation (1 to 5). Tables 1–3 present the various elements related to materials and color under each type of view. Table 4 presents additional events. A number of types of views can be carried simultaneously. The system can also capture the travel distance in which a certain scene or panorama continues to be visible.
- A keyboard operator (usually in the front passenger seat) inputs the remarks of the commentator. Figure 3 shows the keyboard operator with the lap-top computer and the adjacent video camera. Figure 4 shows the coded keys for the input from commentator's remarks. The computer screen shows and holds the quality of a given view until the commentator tells the operator to take it off or scene ends. For example, power poles along a roadway might run for 1 mi or so, the commentator would say, "focal point, man-made, quality 4 (detracts), presentation 2, start now"—a mile later, at end of power poles, "take off power poles."
- An Equipment Operator sits near the VCR, etc., and notes whether all systems are on and functioning properly.

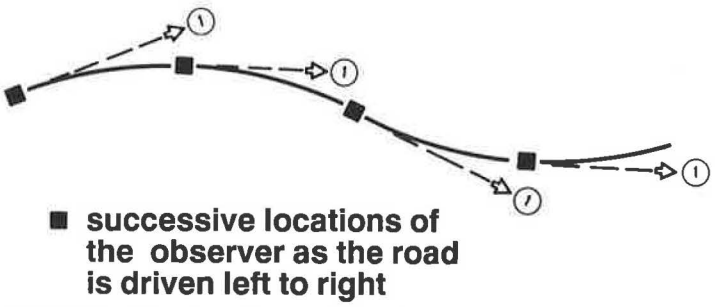


FIGURE 2 Plan view of a curving road showing opportunities for presentation ratings of 1.

TABLE 1 PANORAMA ITEMS RELATED TO MATERIALS AND COLOR

COMPUTER KEYBOARD OVERLAYS		PANORAMA
1		TYPICAL LANDFORM
2		UNIQUE LANDFORM
3		WATER BODY
Q		TYPICAL NATIVE VEGETATION
W		UNIQUE NATIVE VEGETATION
E		TYPICAL AGRICULTURAL
R		UNIQUE AGRICULTURAL
A		MAN-MADE SINGLE STRUCTURE
S		MAN-MADE MULTI-STRUCTURE
D		UNIQUE MAN-MADE
Z		COLOR NATIVE VEGETATION
X		PATTERNS NATIVE VEGETATION
C		MAN-MADE COLOR/PATTERNS

TABLE 2 SCENE ITEMS RELATED TO MATERIALS AND COLOR

COMPUTER KEYBOARD OVERLAYS		SCENE
4		CLIFF/BLUFF/DRAW/DEPRESSION
5		ROCK OUTCROP
6		UNIQUE LANDFORM
7		MOVING WATER
8		WATER BODY
T		VEGETATION EDGE
Y		ISOLATED NATIVE VEGETATION
U		VEGETATION PATTERNS
I		CROPS AND CROP PATTERNS
O		AGRICULTURE NATIVE EDGE PATTERN
F		AGRICULTURAL ACTIVITY
G		AGRICULTURAL STRUCTURES
H		NON-AGRICULTURAL STRUCTURES
J		UNIQUE STRUCTURE
V		COLOR/NATIVE VEGETATION
B		PATTERNS NATIVE VEGETATION
N		MAN-MADE COLOR/PATTERN

TABLE 3 FOCAL POINT ITEMS RELATED TO MATERIALS AND COLOR

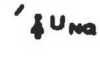
COMPUTER KEYBOARD OVERLAYS		FOCAL POINT
9		LANDFORM EDGE
0		ROCK/ROCK PATTERN
–		UNIQUE LANDFORM
=		MOVING WATER
P		WATER-VEGETATION EDGE
{[		NATIVE VEGETATION EDGE
}]		UNIQUE VEGETATION
\		VEGETATION STRUCTURE EDGE
K		AGRICULTURAL OPERATIONS
L		AGRICULTURAL UNIQUE
:;		MAN-MADE STRUCTURE
M		COLOR NATIVE VEGETATION
,<		PATTERNS/FORMS NATIVE VEGETATION
.>		MAN-MADE COLOR/PATTERN/SYMBOLS
/		MAN-MADE UNIQUE

TABLE 4 ADDITIONAL EVENTS

COMPUTER KEYBOARD OVERLAYS		ADDITIONAL EVENTS
!		TRAVEL ACCOMMODATIONS
@		NATURAL TOURS
#		MUSEUMS
$		REFUGE/NATIONAL LANDS
%		HISTORIC DISTRICT (SITE)
^		HISTORIC/ARCHEOLOGICAL/ETHNIC
&		PARKS/RECREATION AREA
*		REST AREAS
(		ROAD FLOWS WITH TERRAIN
)		ROAD RIBBON
_		ROAD SIDE
+		TRAFFIC

FIGURE 3 Keyboard operator with lap-top computer.

The operator is also responsible for panning the video to record the views that are being described by the commentator. Figure 5 shows the equipment operator with the VCR, etc., in the background.

Table 5 presents a computer printout of the data gathered on a run. In this example, the events were printed out in order of occurrence. The output can also be printed by type of event, by distance, etc.

The collection and recording of information gathered during the evaluation of a potential scenic byway can be very complex and time consuming. One state reported that a team of 4 to 6 persons drove the roads and laboriously recorded

FIGURE 5 Equipment operator.

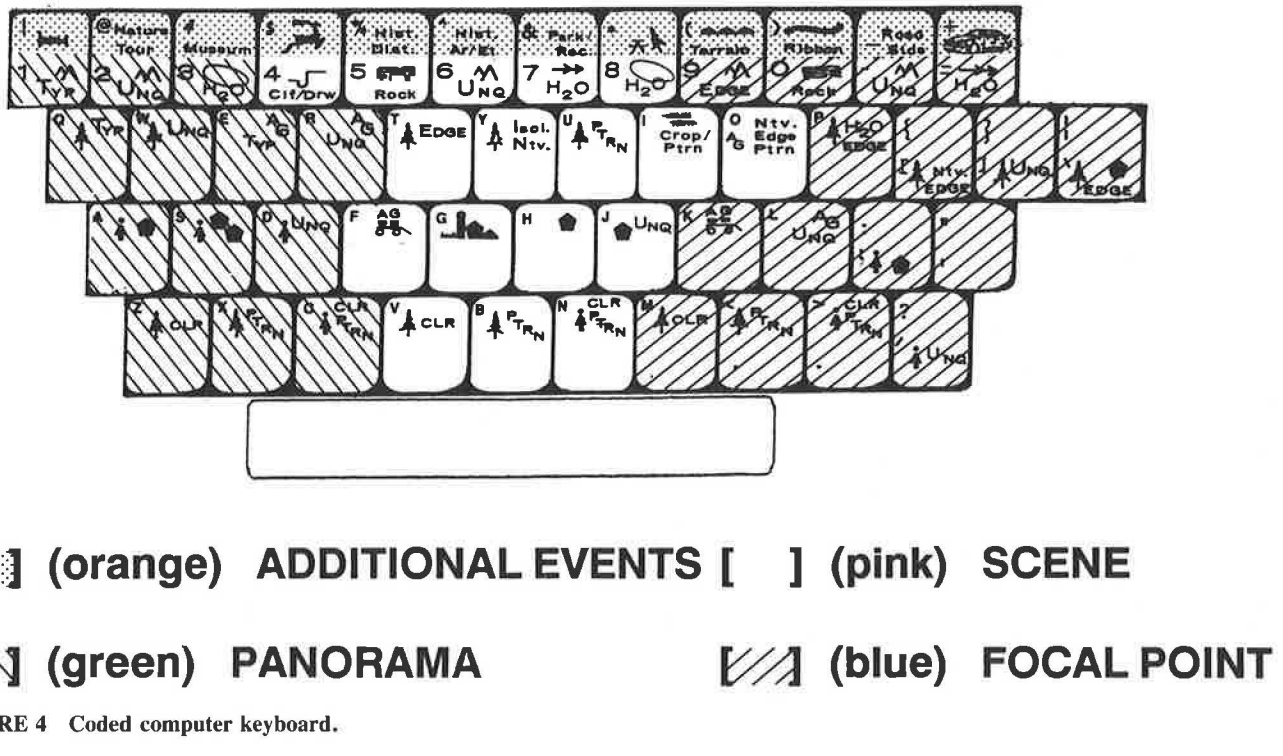


FIGURE 4 Coded computer keyboard.

TABLE 5 PRINTOUT OF DATA

Route 4B
Thu, May 3, 1990
9:25 am
Missouri - 79, PIKE & RALLS COUNTIES
NORTHBOUND FROM SCENIC OVERLK, NO. OF RT. T TO RT. N

Event Seq	Note	Event Code	Quality of View	Quality of Presentation	Distance (ft) Begin	End	Speed (mph)	Time	Event Activity Description
001	*	211	2		000000	031281	00	00:01:06:44	Road ribbon = [Shift] 0
002	*	176	2	3	000000	009956	00	00:00:47:56	S:Vegetation edge = t
003	*	152	2	3	001153	001390	2*	00:02:05:03	P:Water = 3
004	*	170	2	3	002827	003224	19	00:03:12:26	S:Cliff/Bluff/Draw/Depression = 4
005	*	170	2	3	003700	004024	3*	00:03:59:36	S:Cliff/Bluff/Draw/Depression = 4
006	*	216			005870	005870	19	00:05:36:93	Parks/Recreation areas = [Shift] 7
007	*	176	2	3	010256	015102	32	00:07:13:05	S:Vegetation edge = t
008	*	172	2	1	011290	012963	7*	00:08:02:69	S:Unique land form = 6
009	*	190	2	1	011415	012890	9*	00:08:12:61	F:Rock, rock pattern = 0
010	*	172	1	1	012976	015906	4*	00:09:08:44	S:Unique land form = 6
011	*	190	2	2	013399	014085	21	00:09:56:14	F:Rock, rock pattern = 0
012	*	176	2	3	015605	017406	33	00:10:39:62	S:Vegetation edge = t
013	*	204	5	5	016856	031281	32	00:11:04:10	F:Man made color/pattern/symbol = .
014	*	182	4	3	017539	019218	18	00:11:19:21	S:Agricultural structures = g
015	*	172	2	1	020607	022105	25	00:12:46:75	S:Unique land form = 6
016	*	170	2	3	021195	021945	8*	00:13:03:17	S:Cliff/Bluff/Draw/Depression = 4
017	*	170	2	3	022161	022356	1*	00:13:51:03	S:Cliff/Bluff/Draw/Depression = 4
018	*	170	2	3	022383	022954	9*	00:14:15:28	S:Cliff/Bluff/Draw/Depression = 4
019	*	172	2	1	024657	027839	25	00:15:23:66	S:Unique land form = 6
020	*	176	2	3	026511	031281	34	00:16:00:66	S:Vegetation edge = t
021	*	172	1	2	027900	029391	28	00:16:28:18	S:Unique land form = 6
022	*	179	2	3	028134	030989	27	00:16:33:53	S:Crops and crop patterns = i

comments, distances, and locations. Later the recorded notes were summarized for a written evaluation. If the information is to be used in any type of computer program evaluation, the task takes on a whole new level of complexity.

The lap-top computer/DMD/video camera system makes the complex task of collecting and recording the field information a fairly easy one. A system operating team can be trained in the operation and use of the system in 2 to 3 days of training on the road.

In addition to the relative ease of collecting the field data, a further advantage of the computerized system lies in the use of the computer-recorded data for developing a rating number for any road being considered for scenic byway designation.

FUTURE RESEARCH

The system described will be used to gather field information on a number of potential byways as well as some roads with little or no byway potential in each of the four states. The data will be analyzed to develop rating numbers for potential byways. It is expected that a numerical rating system will be developed that can be used to indicate the level "byway quality" of any rated roadway. For example, on a hypothetical scale of 1 to 10, any rating less than 7 might disqualify a road for designation as a scenic byway. On the other hand, a road with a rating less than 7 might have its rating raised to 7 or above if some of the 5 views (distracting or detracting views) such as roadside trash, junk cars near residences, or excessive number of obtrusive billboards were improved or removed through local cleanup efforts.

CONCLUSION

The developed quantitative approach to determining rating numbers for use in the selection and designation of scenic byways will be used first as a Byway Project research tool. At a later time, it will be included as part of the total procedure for use in the four states' activities in future byway designations.

The development of quantitative techniques for determining rating numbers of potential byways is the primary purpose of the selection and designation portion of the Byway Project.

ACKNOWLEDGMENTS

This work was sponsored by the Transportation Departments of Iowa, Kansas, Missouri, and Nebraska and the Midwest Transportation Center, the U.S. Department of Transportation–funded research center for the four-state region, operated by Iowa State University and the University of Iowa. Special thanks are given to Joe Mickes, Missouri Highway

and Transportation Department, for his vision of the need for the study and his assistance as a fundraiser and expediter.

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Management and Maintenance

Rehabilitation and Upgrading Design of Earth and Gravel Roads in Tropical Developing Countries

José Carlos de O. S. Horta

The earth road construction standard that is required for a given traffic volume appears to be a function of the terrain morphology and type of soil. Instead of a formal application of a priori concepts, site investigation of natural conditions and road performance should be a sound basis for design of good-quality and cost-effective rehabilitation and upgrading of earth and gravel roads. An appropriate design methodology based on comprehensive field investigations was developed for road rehabilitation schemes in Madagascar and tested in contract works in that country, and also in Zaire. The methodology proved to save money and time, and rather than resulting in poor design, capturing conditions and resolving design problems in the field may improve the design quality.The methodology comprises three phases. The first phase is the comprehensive inventory and detailed condition survey, the second phase is field design, and the third phase is complementary office design and estimation of quantities. The accuracy of quantity estimates derived from this simplified design methodology appears to be satisfactory for contract rehabilitation works.

The rehabilitation and upgrading of a road presupposes an existing road constructed to certain standards and requiring repair works (rehabilitation) following traffic- and climate-induced deterioration, as well as some improvement of design standards (upgrading) to cope with increasing traffic demands.

Unsurfaced roads show a wide range of construction standards. If the road formation or pavement is considered, the following five design standards or construction stages can be defined:

Standard 0, track beaten by animals and carts;
Standard 1, dust road or track opened and used by motor vehicles;
Standard 2, earth road constructed by labor intensive methods or by scraping with a motor grader;
Standard 3, earth road with supply of selected soils to form a wearing course; and
Standard 4, earth or gravel road with wearing and base courses forming a specific pavement.

At construction Stage 2, clayey pavements will not withstand traffic loads during the rainy seasons. In order to avoid damage to the road pavement, traffic restriction during and after rain is usually recommended but experience has shown that rain gates are often ineffective for social or cultural reasons. Clayey pavement sections should preferably be upgraded to Stage 3.

Av. Bombeiros Voluntários, 42 Algés, 1495 Lisboa, Portugal.

Roads constructed to Standard 2 follow land forms with small-radius curves and steep gradients on hills and mountains. Ground undulations and pavement roughness usually do not allow for high speeds, and the average driving speed is normally low in the range of 20 to 50 km/hr.

At Standard 3, improvements to the road alignment such as embankments on swampy stretches and in places liable to flooding are considered. The average driving speed may range from 40 to 60 km/hr.

For Standard 4, upper gradient and lower radius limits are usually imposed and the road width should preferably be designed for two lanes. The average driving speed may range from 50 to 70 km/hr or more.

Mellier (*1*) suggests that earth roads be upgraded to Standards 2, 3, and 4, respectively, for the following traffic levels: up to 25, up to 50, and up to 200 vehicles (vpd).

Considering drainage and water-crossing would result in some different construction stages, but drainage and water-crossing construction standards can also be correlated with the pavement construction standards.

Water-crossing structures are nonexistent or exceptional for Stages 0 and 1; rivers and streams are forded where and when possible. At Stage 2, crossings are usually timber structures of ferryboats, and later, masonry, concrete, and metal bridges. Drainage structures are progressively constructed between Stages 2 and 4 to avoid road closure and damage by run-off.

Pavement surface drainage (longitudinal and diversion ditches, diversion banks, and thresholds) should be contemplated as early as Stage 2, to control erosion by run-off and avoid fast deterioration of the pavement surface. Most roads constructed or upgraded to Standard 2, and even to Standards 3 or 4 without proper control of run-off, appear to have reverted to lower standards after a few rainy seasons or even a single storm.

Subdrainage and slope stability problems require investigation by qualified professionals and the corresponding works are usually contemplated in late stages.

Most often in practice, in tropical developing countries, a single road is composed of different stretches constructed to different standards. Discontinuous construction actually appears to be well adapted to low-volume earth roads.

Typical tropical soils often show exceptionally good performance under traffic. If a road stretch on a lateritic plateau is carrying traffic without distortion and with slight wear (Figure 1), this stretch can be maintained at Stage 1, while other stretches will have to be upgraded to Standards 2, 3, or even 4, for comparable serviceability.

FIGURE 1 A section of the road Lai to Doba, Southern Chad: the pavement material is undisturbed ferruginous soil and the traffic comprises heavy vehicles for collection of cotton crops and transport of miscellaneous goods. A sample of pavement soil was classified as low-plasticity clay, CL, after laboratory testing and exhibited a soaked CBR of 40 percent at 95 percent modified compaction.

Thus, for instance, in contrast to a road on a lateritic plateau, a road on a swampy, clayey depression will have to be constructed to Standards 3 or 4 even before it can be used by traffic.

Thus, the construction standard that is required for a given traffic level appears to be a function of the terrain morphology and the type of soil. Site investigation of natural conditions and road performance, rather than formal application of a priori concepts should be a sound basis for effective rehabilitation and upgrading design of low-volume earth and gravel roads.

APPROPRIATE DESIGN METHODOLOGY

A design methodology based on comprehensive field investigation of the existing roads was developed in Madagascar (*2*) for application to earth road rehabilitation schemes that were undertaken on force account by the Ministry of Works maintenance gangs under the leadership of specially trained young engineers and technicians. The resulting evaluation of quantities and scheduling of works proved to be accurate and realistic enough for the methodology to be extended to rehabilitation works by contract (Jean Carbonnel, 1987 to 1989, unpublished data on the rehabilitation of 600-km earth roads in southeastern Madagascar).

The methodology was further improved and applied to road projects in other tropical countries, namely the rehabilitation and upgrading design of the road from Kinkondja to Malemba Nkulu, Mwanza Seya, and Musao, a 160-km earth road in Central Shaba, Zaire. Contract works based on this design were afterwards funded by the U.S. Agency for International Development.

The rehabilitation and upgrading design of the road Kinkondja-Musao was completed in 3 months, including specifications and contract documents. It required 2 months of field investigations and 1 month of office work by a team composed of an expatriate engineer, a national soils and materials engineer, and a national survey engineer with some experience of earth roads maintenance. After this project, both national engineers were considered to be trained to carry out similar projects without the assistance of the expatriate engineer.

The equipment required for field work mainly comprised two four-wheel-drive vehicles, camping accessories, two decametric measuring chains, surveyor level, camera, sample bags, shovels, pick-axes, and crowbars. With the exception of drivers, all workers were hired in the field.

The design methodology comprised three phases. The first phase was the inventory and condition survey of the road, including the inventory of culverts and small structures, the inspection of bridges, and the inventory of soils and materials. The tasks comprised in this phase were carried out in the field, except for the testing of possible borrow soil samples that were shipped to a central laboratory in Lubumbashi. However, for large projects, a field laboratory can be contemplated.

The second phase is the design of the maintenance and rehabilitation works required for upgrading. This phase should also be carried out and completed in the field to allow for all necessary checking, collection of further information, and assessment of field conditions.

The third phase is the estimation of quantities after appropriate definition of pay items on the basis of the rehabilitation works resulting from the second phase. This phase is best carried out in the central office with the assistance of computers and technical files.

The rehabilitation and upgrading of the road Kinkondja-Musao was part of a rural development project. Before rehabilitation, this road only carried < 1 vpd. Rehabilitation and upgrading were intended to contribute to the development of commercialization of food crops in a potentially rich area with a relatively dense population.

For this and for most rural roads of developing countries, a conventional design methodology as practiced in developed countries would result in prohibitive costs, the order of magnitude of construction costs, incommensurate with the traffic level. Rather than a drastic decrease in length of road to be rehabilitated to afford the luxury of a conventional design, two options were left: either start the works without design, or apply an appropriate design methodology. The second option appears to have minimized risks and construction costs as well as global service life costs.

INVENTORY AND CONDITION SURVEY PHASE

The preliminary task of this phase is the staking out of the road. Stakes should be put along the road every 50 m to allow for easy location during the field investigations.

The detailed inventory of road features and structures and the condition rating of the pavement and appurtenant features should be made while walking along the road. Depending on terrain and road condition, 5 to 20 km of earth road can be surveyed during a working day walk.

The survey is made by filling a previously prepared form (Figure 2) at scale 1/10 000 (1 mm for 10 m).

In the first box of this form, the linear sketch shall comprise in graphic form all information about junctions, rivers, vil-

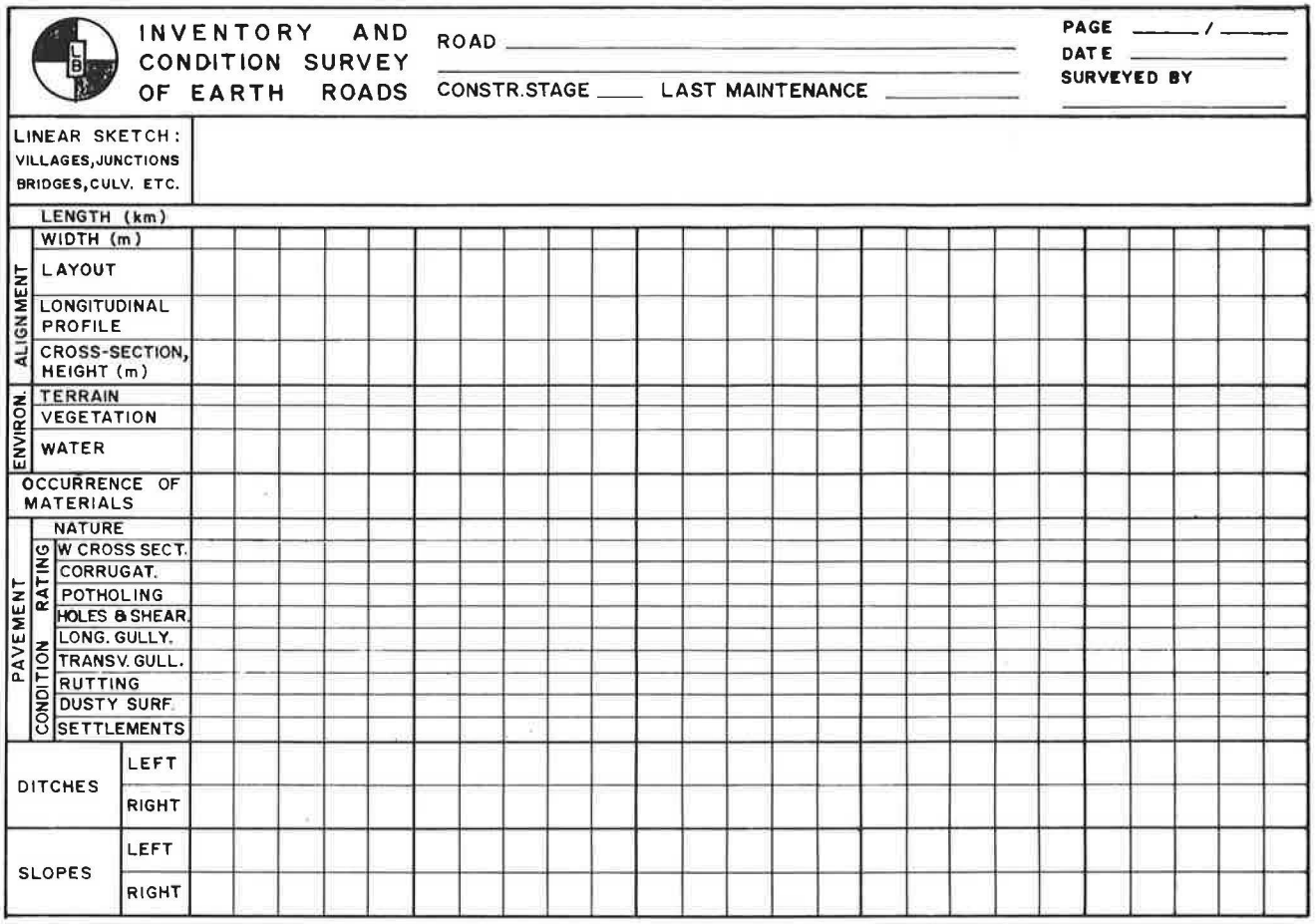

FIGURE 2 Inventory and condition survey form.

lages, as well as numbered symbols indicating the location of drainage structures and water crossings. The former are listed and inventoried with more detail in a separate form (Figure 3), whereas the latter are the subject of special reports after inspection (*3*).

The chainage is inscribed in the second line, and under this line, the form comprises five horizontal sections: road alignment, environment, pavement (type and condition rating), ditches, and slopes.

The first section receives all information relative to width, layout, longitudinal profile, and cross sections collected and estimated without the help of survey instruments. For accurate estimation of earth works, some particular sections, such as important embankments, and additional cuts or rock excavations are separately surveyed by instruments.

The second section of the form is devoted to the environment of the road. In the first-line water surfaces, streams and rivers are positioned with mention of approximate values of level and flow rates as well as flooding and road overtopping evidence. The type and density of vegetation and crop-yielding areas are inscribed in the second line by means of suitable abbreviations or symbols. In the third line, the terrain is characterized by the usual rating (respectively 1 to 5 for flat, rolling, hilly, mountain, and steep mountain terrain) and by additional symbols for particular forms of landscape, such as scarps, erosion gullies, anthills, etc.

The line on materials occurrences comprises mention of superficial occurrences and outcrops as well as occurrences in pits dug along the road. The field identification symbol is noted and, where possible, estimated or measured CBR values as well as estimated minimal volumes in hundreds or thousands of cubic meters are given. This line may receive additional information after laboratory testing of selected soil samples from trial pits.

The third section of the form is devoted to pavement condition rating; it is discussed in more detail later.

In the two lower sections of the form, ditches and important cut and embankment slopes are positioned and rated. Ditches may be missing (Rating +2), obstructed by debris and vegetation (Rating +1), in good condition (rating 0), or show different depths of erosion (ratings 1, −2, and −3, respectively). Slope erosion may require minor repairs by hand and turfing (Rating 1), major repair with borrowed materials (Rating 2), or reconstruction (Rating 3). Slope instability usually requires major repair or reconstruction as well as drainage and retaining works. Specific cases of slope instability men-

INVENTORY OF CULVERTS AND SMALL STRUCTURES

ROAD _____

PAGE ____/____
DATE _____
SURVEYED BY_____

N°	KM	TYPE OF STRUCT.	SECTION (cm)	LENGTH (m)	INLET	OUTLET	CONDITION

FIGURE 3 Form for inventory of drainage structures.

tioned in the inventory and condition survey form should be the subject of detailed inspection reports with descriptive sketches and pictures.

EARTH PAVEMENT CONDITION RATING

The pavement condition rating is an important task of the first design phase. It requires identification, objective rating and interpretation of pavement distress and deserves special discussion.

Two agents are responsible for earth road pavement deterioration: water and traffic. In tropical countries, water is often the main agent (*4*).

Water flowing along steep gradients and for long distances digs erosion gullies on pavement surfaces. Longitudinal gullying (Figure 4) along steep gradients shows sharp V-shaped cross sections and increasing depth with increasing length. Transverse gullies along the cross-fall usually keep small dimensions and are mainly located in curves at the vicinity of the inner edge. The outlets of longitudinal gullies across the pavement are not considered as transverse gullies. Transverse gullies are often repeated like corrugations (Figure 5).

Traffic-induced distress of earth pavements may be either functional or structural.

Functional distress results from the wearing action of vehicles. Soil elements are pulled out by vehicle tires, with loss of fines into the atmosphere and displacement of coarse elements. Functional earth pavement distress takes the form of W cross sections, corrugations, and potholing.

As traffic wearing develops with low densities of vehicles running in the middle of generally narrow pavements, soil

FIGURE 4 Longitudinal gullying.

FIGURE 5 Transverse gullies.

FIGURE 7 Low-degree W cross section, Southern Madagascar.

elements are thrown to the shoulders and the center of the pavement. This effect together with differential wearing along the wheel paths results in a flat *W*-like cross section. This type of distress is therefore called "W cross section" (Figures 6 and 7) and should not be taken for rutting.

An additional effect of traffic wearing on gravel pavements is the development of corrugations (Figure 8). After breaking of cohesion bonds during dry weather, coarse sand and gravel particles are brought together into transverse ripples with a regular spacing of about 1 m in most cases as a result of heavy vehicles' vibration and pull up by the tires (*5*).

Potholes mainly result from pull out of single coarse elements; thus, potholing is uncommon on sandy and fine soil roads, but typical of gravel and crushed-stone lined pavements (Figure 9).

Structural distress is a result of insufficient soil strength to carry the traffic loads. Soil shearing under traffic seldom results in local shears and usually produces rutting (Figures 10 and 11). Ruts develop longitudinally as the soil is expelled by the wheels and forms lateral rolls. Rutting characterizes soils with

FIGURE 8 Corrugations of a lateritic gravel pavement, road Kelo to Moundou, Chad; W cross section is also apparent.

FIGURE 6 W cross section, Kilometer 10 from northern end, National Road 44, Madagascar; the hollow segments of the cross section are particularly sharp as a result of wearing by iron fellies of bullock carts.

FIGURE 9 Potholing of a well-graded crushed-stone pavement, National Road 5A, Madagascar.

FIGURE 10 Rutting of dry cohesionless sand, Kilometer 131, National Road 35, Madagascar.

FIGURE 12 Dusty surface of fine soils, Kilometer 55, Kelo-Lai road, Chad.

FIGURE 11 Rainy season rutting of a recently rehabilitated earth road, Bandundu province, Zaire.

FIGURE 13 Holes and shearing, Kilometer 35, National Road 10, Madagascar.

low bearing capacity, namely wet fine soils and dry noncohesive, poorly graded sands, as well as poorly compacted soils.

Dry fine soils show an additional particular type of structural distress resulting from breaking of cohesion bonds by traffic-induced vibrations. The upper layer of soil changes into dust with low unit weight, covering and concealing ruts, holes, and surface roughness (Figure 12).

The combined action of seasonal water ponding and traffic wear results in large holes and shearing (Figure 13). This type of distress is typical of sections in which water crosses the pavement and drainage structures or appropriate pavement protection is missing.

Some particular types of soils may develop shrinkage cracks after compaction and during the dry season. However, cracking is neither a significant nor a widespread type of distress on earth and gravel pavements. Where present, as for instance, in clay road pavements of arid regions, it takes the form of alligator and block cracking and originates potholing.

In addition to run-off water and traffic-induced pavement distress, settlements should also be mentioned. Settlements result from slope or earthwork instability.

Any type of pavement distress should be characterized by its extent and degree of severity (*6*). The nine lines under "Condition Rating" in the inventory and condition survey form (Figure 2) allow for the different, possible types of distress (*W* cross section, corrugations, potholing, holes and shearing, longitudinal gullying, transverse gullies, rutting, dusty surface, settlements) to be represented at the exact sections and areas where they occur. Objective rating of the different types of distress is done (see Table 1) by taking measurements and evaluating distress severity parameters such as depth, amplitude, and density. The degree of severity of a given distress type is indicated in the condition survey form by different types of hatching of the relevant extent areas. Digits indicating the degree of severity may also be used for this purpose. Four degrees of severity were considered, but some distress types would only require three degrees for adequate rating.

It is important that the limits between degrees of severity be set in such a way as to characterize the different maintenance and rehabilitation techniques, as described in the next section.

The comprehensive pavement condition survey and rating are effectively an evaluation of soil behavior under traffic that can be considered as more appropriate and secure than conventional laboratory testing. Using this survey and rating sys-

TABLE 1 SEVERITY RATING OF EARTH ROAD DISTRESS

Types of distress	Measurement	Severity rating			
		0. Not detectable	1. Appreciable, possible consequences	2. Important, harmful consequences	3. Utmost, very harmful consequences
W cross-section	depth (cm)	< 5	5-10	> 10	-
Corrugations	amplitude (cm)	< 2	2-5	> 5	-
Potholing	surface (%)	< 5	5-20	20-50	> 50
Holes and shearing	maximum depth (cm)	< 5	5-10	10-25	> 25
Transverse gullies	depth (cm)	< 2	2-5	> 5	-
Longitudinal gullying	depth (cm)	< 5	5-10	10-25	> 25
Rutting	amplitude (cm)	-	5-10	10-25	> 25
Dusty surface	surface (%)	< 1	1-5	5-20	> 20
Settlements	difference in level (cm)	< 5	5-10	10-25	> 25

tem, an experienced geotechnical engineer will be able to identify and characterize the pavement soils with accuracy, and do without laboratory testing of formation soil samples. Collection of samples and laboratory testing may therefore be restricted to the identification of possible borrow areas.

FIELD DESIGN PHASE

A well-formulated problem is a potentially solved problem, and in this sense an accurate inventory and condition survey can be considered as equivalent to rehabilitation design. Specific requirements for rehabilitation works can easily be derived from the inventory and condition survey form for each road section and reported in a field design form (Figure 14).

The field design form comprises several lines corresponding to the road alignment, pavement, ditches, structures, and slopes.

Modification of layout of an earth road should always be contemplated at the time of rehabilitation. Earth roads represent only a small capital investment. Alternative layouts avoiding eroded or low-bearing-capacity sections, as well as sections that proved to be difficult to drain, almost certainly result in lower rehabilitation and maintenance costs. Sometimes after a few maintenance scraping operations, the road becomes entrenched (Figure 15) with long diversion ditches, and in this case the construction of a new earth road alongside the existing one will prove to be the best choice.

The rehabilitation design sometimes requires additional cuts to avoid eroded slopes and often the existing embankments require reconstruction and raising, following erosion by traffic and water. Quantities for small earth works may generally be evaluated with acceptable accuracy without surveying, but preferably, sections requiring important earth works should be surveyed.

Pavement works include light grading or blading without scarification by a motor-grader; heavy grading I, comprising scarification and blading by a motor-grader and final compaction; heavy grading II, if a bulldozer is required in addition to the motorgrader; and overlaying.

Overlaying with selected materials is required for sections showing rutting as well as for sections where low-bearing-capacity soils were previously overlaid, in order to compensate for materials loss under traffic. The overlay thickness is usually designed as a function of traffic and bearing ratio of the formation soil (*7*).

Light and heavy grading I and II are required to repair run-off water induced as well as traffic functional distress showing the Degrees of Severity 1, 2, and 3, respectively (Table 2). Heavy grading II with an additional bulldozer is also required for sections with rock materials.

Miscellaneous pavement works include protection against run-off erosion of sections where the gradient exceeds the critical erosion value for the relevant soil. This can be done by gravel overlaying, stone lining, or chemical stabilization, as well as by diversion banks and thresholds.

Ditch rehabilitation works include excavation, cleaning, and protection against erosion.

Rehabilitation works on drainage structures include maintenance, rehabilitation, and construction with reference to standard design plans.

In the last line of the rehabilitation design form, works for slope stabilization, including tree planting as well as protection against erosion including turfing or grassing, may be specified for relevant sections. Tree planting and grassing are normally the most cost-effective works for slope stabilization and protection, but given sections may require sustaining gabions or other types of works, together with specific design drawings.

ESTIMATION OF QUANTITIES

After field work, the first task of Phase 3 is the identification of adequate pay items consistent with the different works considered appropriate for rehabilitation. Most of the pay items, including earth works items, do not differ from usual construction practice, but for earth or gravel pavements, light grading, heavy grading I and II (measured by length or sur-

REHABILITATION FIELD DESIGN

ROAD _____

PAGE ____ / ____

DATE _____

LINEAR SKETCH OF EXISTING ROAD		
LENGTH (km)		
LAYOUT		
CUTS		
EMBANKMENTS		
PAVEMENT	L. GRADING	
	H. GRADING I	
	H. GRADING II	
	OVERLAYING	
	MISCELLANEOUS	
DITCHES	EXCAVATION	
	CLEARING	
	MISCELLANEOUS	
STRUCTURES	MAINTENANCE	
	REHABILITATION	
	CONSTRUCTION	
SLOPES		

FIGURE 14 Field design form.

FIGURE 15 A poorly drained entrenched section of an earth road resulting from periodic maintenance by scraping has been deviated by traffic.

face), and overlaying with selected materials (measured by volume), prove to be practical and effective.

The estimation of quantities of works made on the basis of field inventories and condition surveys completed by selective instrument survey proved to be well within a limit of ±20 percent, although certain items, particularly diversion ditches, were poorly evaluated and required exceptions to the contract clause on variation of quantities.

If construction is delayed through a wet season, a new condition survey may be required to update quantities of works.

CONCLUSIONS

A simplified design methodology for rehabilitation and upgrading of low-volume earth and gravel roads based on field work, namely a comprehensive inventory and detailed condition survey by filling forms while walking along the road, proved to result in accurate quantity estimations and low-cost design.

This methodology does not require exhaustive surveying. Instrument surveying may be restricted to cross sectioning of certain sections for accurate estimation of important earth works. Soil testing is restricted to investigation of possible borrow areas as the methodology includes objective rating and interpretation of earth pavement distress.

The design methodology is considered to be most appropriate to developing countries where financial resources are scarce and low traffic volumes do not justify conventional design methodologies. The methodology proved to save money

TABLE 2 PAVEMENT REHABILITATION WORKS AS DERIVED FROM TYPE AND SEVERITY OF DISTRESS

Type of distress	Degree of severity			
	0	1	2	3
Functional traffic and run-off induced distress	No rehabilitation required	Light grading (only motor-grader blade)	Heavy grading I (scarification and blading by motor-grader and final compaction)	Heavy grading II (scarification by bulldozer, blading by motor-grader and final compaction)
Structural distress (rutting or dusty surface)	Overlaying			

Remark: Overlaying of previously overlaid sections could also be required for compensation for traffic induced materials losses and those sections would not necessarily show structural distress prior to rehabilitation.

and time, does not require costly or sophisticated equipment, and is well within the capacity of trained and dedicated road engineers. Rather than resulting in poor design, direct recognition of design problems and conditions in the field may improve the quality of design.

ACKNOWLEDGMENTS

The development and testing of the concepts reported in this paper were actively supported by Victor Ramahatra, formerly Minister of Works (and presently Prime Minister) of Madagascar, a highly qualified army engineer who welcomes new concepts, provided they can assist development.

Thanks are also due to Jean Carbonnel, ERG Consult, France, and Richard Wyatt, Brazil, for valuable information and useful discussion.

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Managing Low-Volume Road Systems for Intermittent Use

JERRY ANDERSON AND JOHN SESSIONS

In some areas of the United States, particularly in gentle topography, closely spaced road systems have been developed that, although low cost per mile to construct, are costly in total to maintain. These low-volume roads are often characterized by short periods of use separated by long periods of little or no use. During periods of little or no use, periodic maintenance must be done both to maintain public safety and to prevent degradation. Closing a road requires an investment to prevent unplanned entry and also often requires modification of drainage facilities. Opening a closed road involves an investment. Managers must decide which roads should be left open and maintained, which roads should be closed, which roads should be reopened, and in what period. Managing roads for intermittent use differs from classic transportation development where the emphasis is on construction and transport costs and does not include intermittent road management options and costs. An efficient mathematical formulation for intermittent road management and a numerical example are given.

As low-volume road systems for natural resource development areas become more extensive, a problem becomes how to manage these systems. Typically, in timber harvesting regions even during periods of use, natural resource development roads may average less than 100 vehicles per day, and then for periods of several years or more, they may have no planned use. In some areas, more roads have been developed than will be needed during any one period of activity. If all roads are left open, for purposes of public safety and to prevent damage to the road, some level of periodic road maintenance must be done. Alternatively, roads could be closed through uses of gates, temporary earth embankments, or other barriers, and drainage structures could be modified to prevent damage from water.

Managers must decide which roads should be open in each time period and which roads should be closed. Considerations include the cost of opening roads, closing roads, maintaining roads while they are open, maintaining roads while they are closed, and transportation costs over the alternative routes between entry points in the road network and timber destinations during each period of activity. Although for some roads, the decisions can be made independently of other roads, often, they cannot. In general, a transportation schedule may involve several entry points into a road network during each period, and alternative routes to destinations may exist. Therefore, the planned activities may have to be considered simultaneously.

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The intermittent road management problem differs from the usual road development problem in that in the latter problem only costs of development, maintenance during periods of use in proportion to the level of the activity, and transportation costs are considered. Road openings, closings, and costs that occur during periods of nonuse are not considered.

One way in which to analyze the intermittent road management problem is to use mixed integer linear programming (MIP). In the following sections, the problem is formulated and a numerical example is provided.

FORMULATION

From a known schedule of entries into an existing road system, the problem is to determine which roads are to remain open and which are to be closed and for how long. The objective is to minimize the discounted cost of transportation costs, road opening costs, road closing costs, and road maintenance costs. By minimizing discounted costs is meant to minimize the sum of truck transportation, road opening, road closing, and road maintenance costs over a specified time horizon taking into account the time value of money.

The objective function becomes

$$\text{Minimize} \sum c_{it}x_{it} + \sum u_{it}B_{it} + \sum v_{it}G_{it} + \sum w_{it}D_{it} \qquad (1)$$

where

c_{it} = discounted unit cost of transport and variable road maintenance for Road i in Period t;
u_{it} = discounted cost of fixed road maintenance for Road i in Period t if it is open;
v_{it} = discounted cost of closing Road i in Period t;
w_{it} = discounted cost of opening Road i in Period t;
x_{it} = volume of traffic units using Road i in Period t;
B_{it} = indicator of period Road i is open, where $B_{it} = 1$ in all periods Road i is open, and $B_{it} = 0$ at all other times;
G_{it} = indicator of period Road i is to be closed, where $G_{it} = 1$ in period Road i is to be closed, and $G_{it} = 0$ at all other times; and
D_{it} = indicator of period Road i is to be opened, where $D_{it} = 1$ in period Road i is to be opened, and $D_{it} = 0$ at all other times.

There are four kinds of relationships (constraints) in the problem—relationships to (a) conserve flow at each node in

the road network, (b) to signal which period the road is open, (c) to signal which period the road is to be closed, and (d) to signal which period the road is to be opened. It is necessary to declare that only one type of variable, B_{it}, is an integer variable, with a value of 0 or 1. The variables G_{it} and D_{it} are continuous variables that will be forced to act as integer variables using the relationships discussed next.

Conservation of Flow

Because all traffic that enters a road node (junction) in Period t must exit the node, it is required that

$$\sum x_{kt} = \sum x_{jt} \qquad \text{for all nodes and time periods} \qquad (2)$$

where x_{kt} = traffic entering node in Period t and x_{jt} = traffic leaving node in Period t.

There will be one equation for each node in the road network for each period of the analysis.

Signal When Road Is Open

To do this, an integer variable, B_{it}, is assigned that must be equal to 1 in all periods the road is open, and 0 otherwise. This is done by requiring that B_{it} must be 1 if there is traffic over the road segment in Period t by the relationship

$$MB_{it} \geq x_{it} + x_{i't} \qquad \text{for all } i \text{ and } t \qquad (3)$$

where M is a number sufficiently large to guarantee that if there is traffic over a road segment ($B_{it} = 1$) that M will be larger than the total flow over the segment in both directions. $X_{i't}$ indicates the possibility of flow in either direction over the link. Equation 3 also permits the possibility that the road could be open with no traffic.

Signal When Road Was Open, But Now It Is Time To Close It

For the first period, because the roads were all assumed to be open at the beginning of Period 1, it is required that

$$B_{i1} + G_{i1} \geq 1 \qquad \text{for all } i \qquad (4)$$

If the road is open, B_{i1} will be 1 and the objective function will force G_{i1} to be 0 to minimize costs. By this is meant, that if B_{i1} is 1, then Equation 4 is satisfied regardless of what positive value G_{i1} takes. Because G_{i1} is not necessary to satisfy Equation 4, the linear programming algorithm will never include it as it only adds cost to the objective function. If B_{i1} is 0, then G_{i1} will be 1 because the sum of B_{i1} and G_{i1} must be greater than or equal to 1. It is not possible to have the road open and apply closing costs at the same time. For subsequent periods, it is required that

$$B_{i,t+1} + G_{i,t+1} \geq B_{it} \qquad \text{for all } i \text{ and } t \qquad (5)$$

This expression means that if the road was open in the previous period, then it must either be kept open in the next period or the closing costs must be applied. Because B_{it} and $B_{i,t+1}$ must be 1 or 0, the objective function will force $G_{i,t+1}$ to be 0 when it can be, or at most equal to 1 to satisfy Equation 5.

Signal When Road Was Closed, But Now Is Time To Open It

If a road will be open in this time period, but it was closed in the previous time period, then the road must be opened. This is done by requiring that

$$D_{it} + B_{i,t-1} \geq B_{it} \qquad \text{for all } i \text{ and } t \geq 2 \qquad (6)$$

If the road was open in the previous period ($B_{i,t-1} = 1$), and is to remain open in this period ($B_{it} = 1$), then the objective function will force D_{it} to be 0. On the other hand, if the road was not open in the previous period ($B_{i,t-1} = 0$) and is to be open in this period, then the objective function will force D_{it} to be equal to 1.

These complete the relationships necessary to ensure that the roads are open when they must carry traffic and that roads can be closed when there is no traffic. The objective function of Equation 1 is the criterion that is used to determine which roads are open, which are closed, and for how long.

EXAMPLE

A 15-road segment, 10-node, 3-period problem is used to demonstrate what has been discussed (Figure 1). All roads are initially assumed to be open. Closing costs, opening costs, and periodic (fixed) road maintenance costs required when the road is open are presented in Table 1. To simplify the presentation, these costs are assumed equal for all road segments (links). The costs have already been discounted to the present from the middle of the period at which they occur. Nodes where traffic enters the network and the period of entry are presented in Table 2. All traffic exits the network at Node 10. The discounted transport costs to transport one truckload over each segment during each period are presented in Table 3. In this example, there is no difference in cost for a truck to go in either direction over a road segment. Maintenance costs proportional to traffic have been included in the transport rate.

The equations corresponding to Equations 1–6 were formulated and the problem solved using the linear programming software LINDO (*1*). The optimal value of the objective function was $273,580. The open-road segments for each time period using the optimal strategy are shown in Figure 2. The road segment from Node 1 to Node 2 is left open in Time Period 2, whereas the road segment from Node 4 to Node 5 is closed. If road opening and closing costs had not been considered simultaneously with the choice of transport routes, the solution could have differed. For example, if road opening and closing costs are ignored, the problem re-solved, and then closing and opening costs are manually added as indicated by the solution of the revised problem, the solution value is

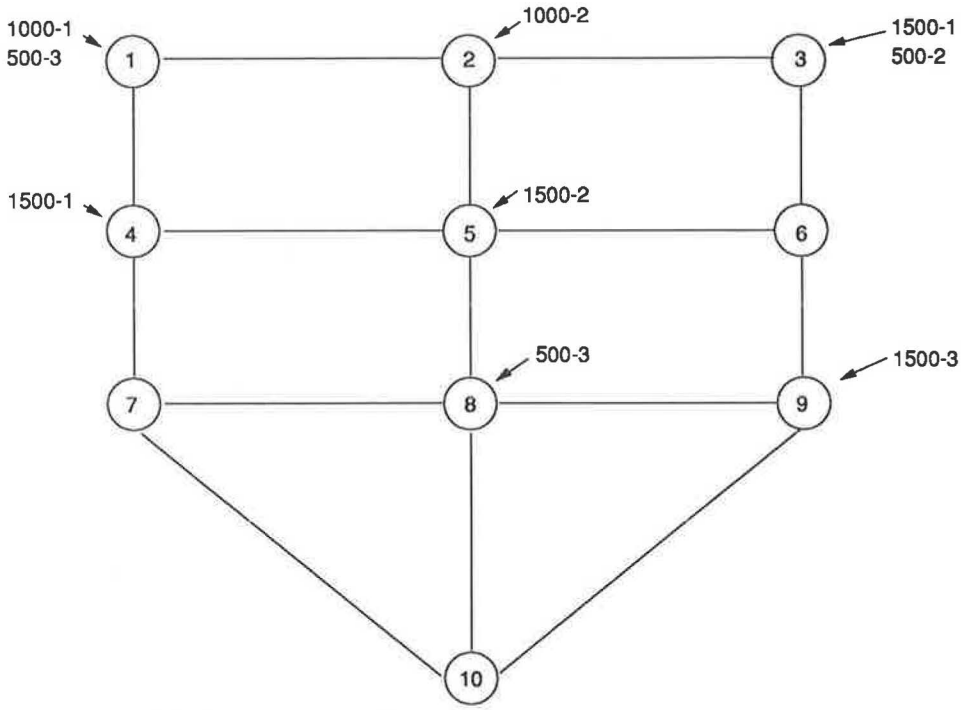


FIGURE 1 Existing road network—the first number indicates the number of truck loads entering the network; the second number is the time period.

TABLE 1 FIXED COSTS FOR OPENING, CLOSING, AND MAINTAINING ROAD LINKS

Period	Link	Opening Cost $	Closing Cost $	Fixed Main. Cost $
1	each	9000	2000	7500
2	each	7397	1644	6164
3	each	6080	1351	5067

TABLE 2 TRUCK LOADS ENTERING ROAD NETWORK IN EACH TIME PERIOD

Truck Loads	Node	Period
1000	1	1
1500	4	1
1500	3	1
1000	2	2
500	3	2
1500	5	2
500	8	3
1500	9	3
500	1	3

$310,161, for an increase of approximately 13 percent. The increase of $36,581 is caused by solving the transportation problem without simultaneous consideration of opening and closing costs. Naturally, the solution is then to minimize transport costs. The open road segments for the revised problem are shown in Figure 3.

PRACTICAL CONSIDERATIONS

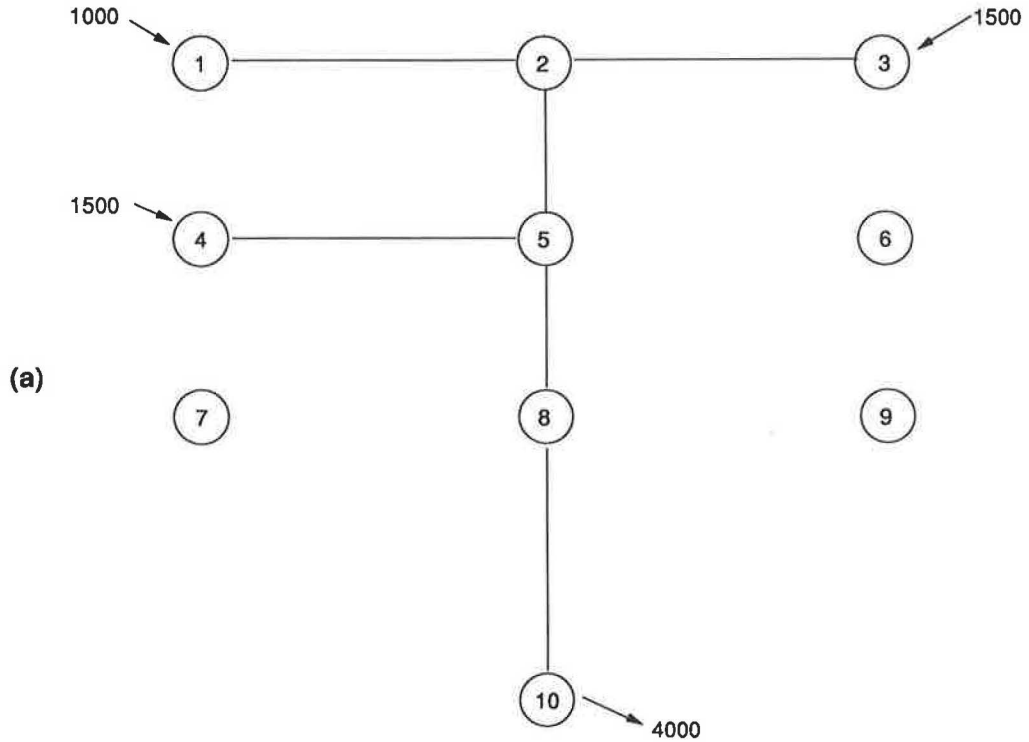
There are several considerations an organization will have to weigh before using this technique to solve road closure problems. As with any numerical technique, good cost data are required. Computing facilities are also a consideration. Computing time goes up rapidly when mixed-integer problems exceed 50 integer variables. Depending on the complexity of the problem and the speed of the computer, computing time may be prohibitive. For problems larger than 50 integer variables, at least a 25-Mhz 80386 or equivalent microcomputer with a math coprocessor is recommended.

Several techniques can be used to reduce solution time:

1. Reduce the number of integer variables. Each 0,1 integer variable can double the possible number of solutions, therefore doubling the solution time required. However, not all road segments need to be integer variables. For example, if a road segment from Node 1 to Node 2 (Link 1–2) in Period 3 will be open regardless of other road management decisions, then add the constraint equation $B_{123} = 1$ and do not declare B_{123} to be an integer variable. Record all road segments that will definitely be open or closed in this manner.
2. Prepare a tight formulation. MIP solution time can be reduced by preparing a tight formulation. Integer program-

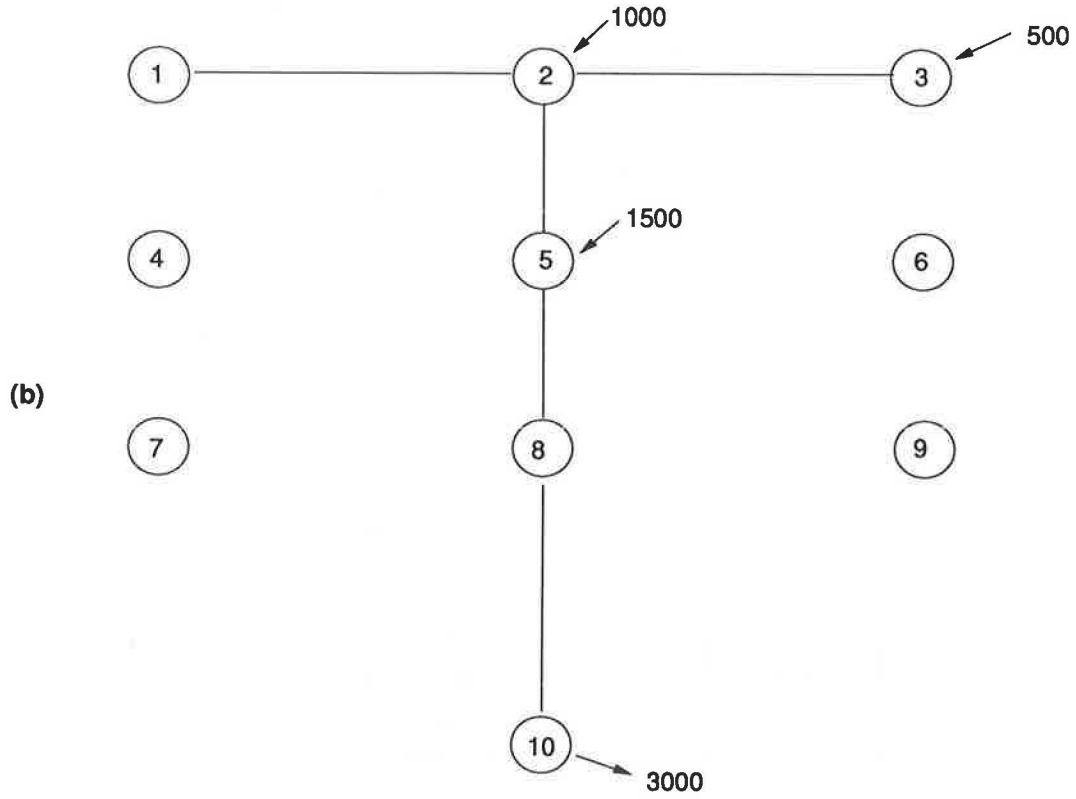
TABLE 3 DISCOUNTED SUM OF TRANSPORT PLUS VARIABLE MAINTENANCE COSTS PER TRUCK LOAD PER LINK

Link	Period 1 $/Load/Link	Period 2 $/Load/Link	Period 3 $/Load/Link
1-2	8.00	6.58	5.40
1-4	7.00	5.75	4.73
2-3	10.00	8.22	6.76
2-5	4.00	3.29	2.70
3-6	7.00	5.75	4.73
4-5	5.00	4.11	3.38
4-7	9.00	7.40	6.08
5-6	8.00	6.58	5.40
5-8	6.00	4.93	4.05
6-9	6.00	4.93	4.05
7-8	10.00	8.22	6.76
7-10	5.00	4.11	3.38
8-9	9.00	7.40	6.08
8-10	6.00	4.93	4.05
9-10	8.00	6.58	5.40



(a)

FIGURE 2 Open road segments resulting from simultaneous consideration of transport, road maintenance, and opening and closing costs for (*a*) Period 1, (*b*) Period 2, and (*c*) Period 3. Total cost of transport, road maintenance, and opening and closing road links is $273,580. (*continued on next page*)



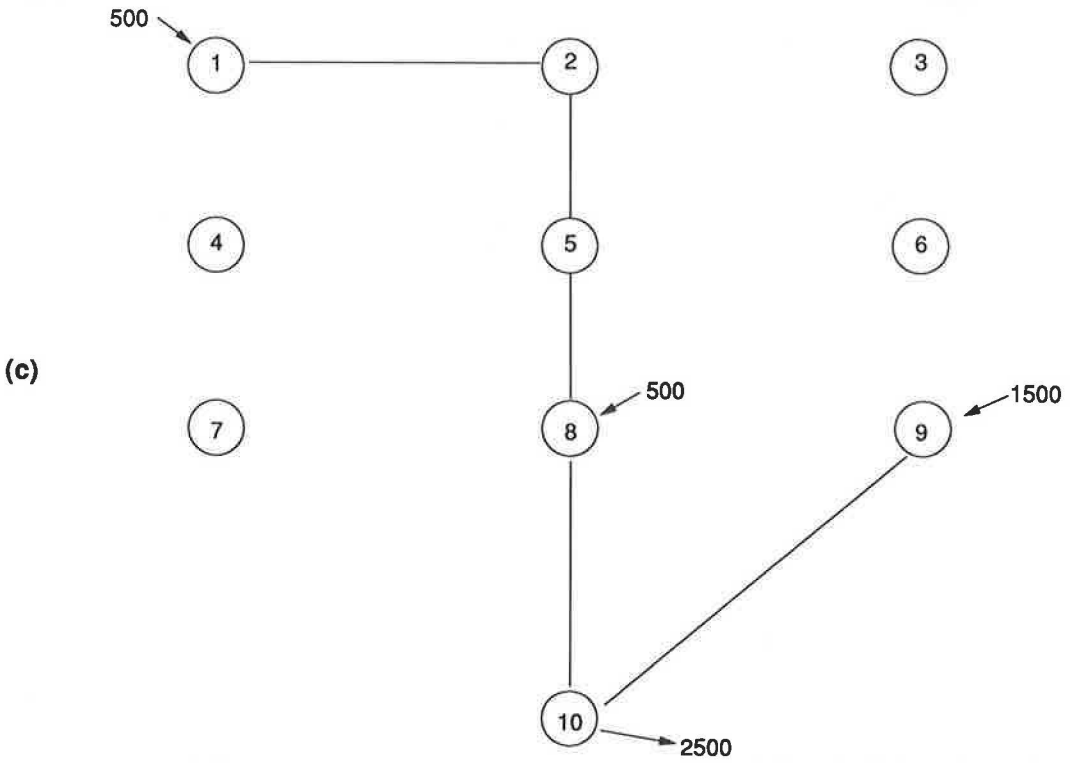


FIGURE 2 *(continued)*

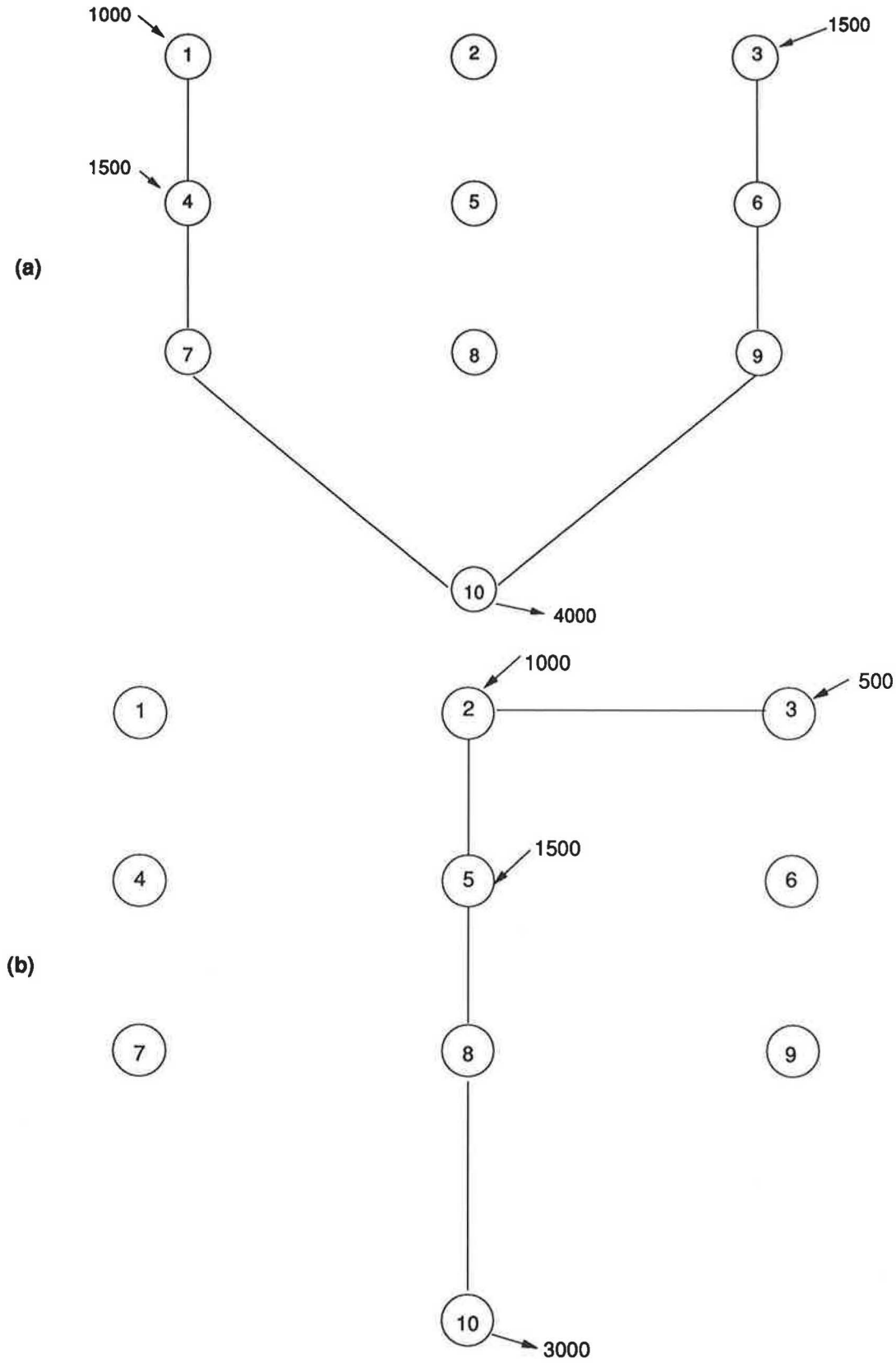


FIGURE 3 Open road segments if opening and closing costs were not considered simultaneously with transport and road maintenance costs for (*a*) Period 1, (*b*) Period 2, and (*c*) Period 3. Total cost of transport, road maintenance, and opening and closing road links is $310,161, a 13 percent increase over the optimum solution. (*continued on next page*)

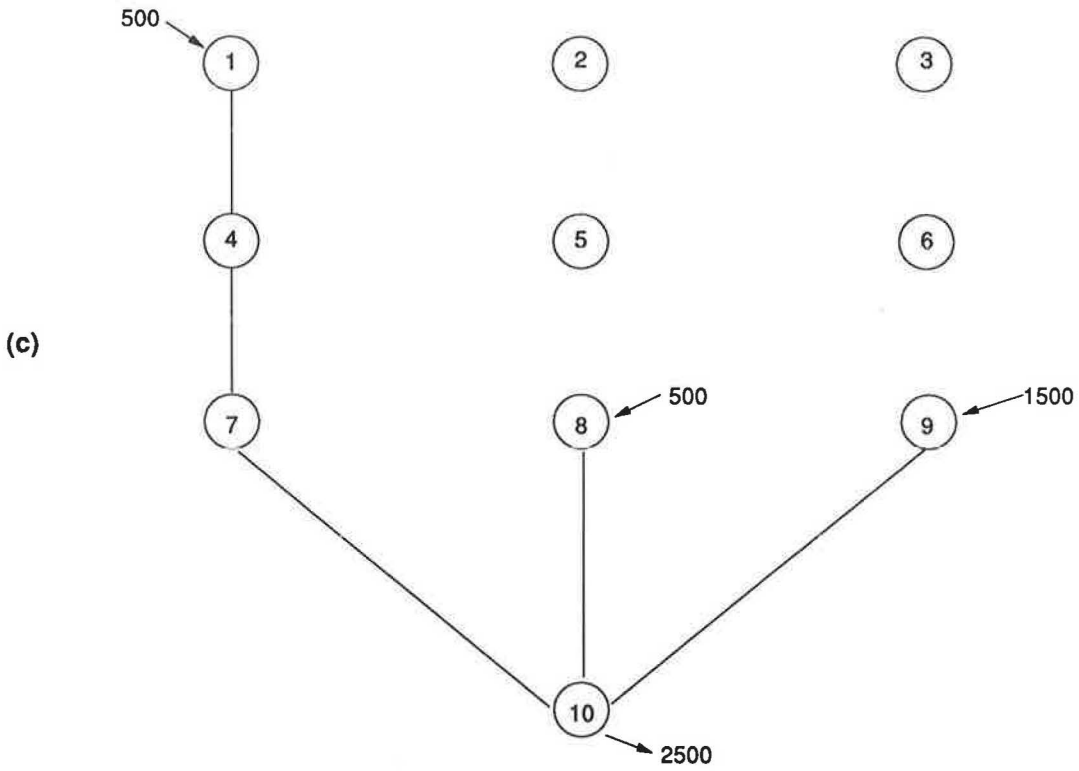


FIGURE 3 *(continued)*

ming packages solve MIPs by first solving the problem as a linear problem (LP), then by iterating the integer variables to solve the MIP. Without a discussion about integer programming algorithms, it suffices that if the initial LP solution is close to the final MIP solution, the formulation is said to be tight. The formulation can be tightened by using a logical large number M in Constraint Equation 3. A number that is too small will make the formulation infeasible when the appropriate B_{ijt} variables are set to 1. However, a number that is extremely large will allow the B_{ijt} variables to take on very small values during the LP solution. This causes the LP solution to be further from the MIP solution, creating a loose formulation and causing a longer solution time. The smallest possible number to use and still ensure feasibility is to set M equal to the total volume entering the road system during that period.

3. Place bounds on the solution. The number of possible solutions to an MIP formulation is 2 to the Nth power, where N is the number of integer variables. To avoid testing all possible solutions in full, an integer programming algorithm using "branch and bound" techniques will examine a solution set until it is clear this set, or branch, is not the optimal one. When the algorithm reaches this point, it will pursue a different branch. Usually, a bound can be placed on the solution value so the MIP will not be pursued beyond this value. A reasonably good bound can save considerable computing time. Three methods for determining good bounds are (a) past experience; (b) run the program, look at the first feasible solution, estimate how much could be saved from this solution, then rerun the program with a revised bound; and (c) use a heuristic procedure to find a good feasible solution and use this as the lower bound.

One final consideration is the type and size of road system being analyzed. This formulation assumes the road system in question is large enough that optimum management strategies are not obvious. Some systems managed by small landowners may have few alternatives where such a complex formulation is not required. Other road systems may have characteristics that do not fit in well with this type of formulation. An example would be extremely broken ownership in which the landowner may not have control over opening and closing the majority of roads.

CONCLUSION

The preceding formulation is one technique to create management strategies for intermittent road use. It may be modified to account for other operational characteristics such as capacity constraints. For applications in which more than 50 integer variables are involved, a 25-Mhz 80386 microcomputer or equivalent with math coprocessor is recommended. All attempts should be made to reduce the number of integer variables. Remember that each integer variable can double the number of possible solutions and double the computing time. Having a tight formulation and placing reasonable bounds on the solution can also reduce computing time considerably. Large areas should be divided into logical road system boundaries where no more than 500 integer variables exist. As faster computers become available, larger problems can be solved with the formulation presented here.

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Iowa Design Exception Process: Application of Benefit-Cost Analysis for Restoration, Rehabilitation, Resurfacing, or Reconstruction of Secondary Roads

CLETUS R. MERCIER, LARRY R. JESSE, AND LOWELL E. RICHARDSON

The problem of providing a rational procedure for allowing exceptions to current design guides for secondary roads is discussed. Many paved secondary roads in Iowa are in need of considerable work, mostly as rehabilitation, resurfacing, and reconstruction. Some will need considerable investment to bring them up to current design guides, sometimes when the amount of investment cannot be economically justified. The Iowa Design Exception Process, a rational decision process for determining the bases for allowing exceptions to meeting current design guides, is described. Information is also included on how the process has worked in Iowa, and a case study is discussed to examine some of the issues arising during the application and review process. The procedure has proven to be a success. What remains lacking is a court test of the procedure. As of the date of writing, no tort claim has been made on the basis of exceptions granted to the current design guides.

In 1947, the Iowa General Assembly passed a farm-to-market (FM) road bill that authorized the designation and establishment of an FM system to include not more than 35,000 mi of secondary roads. Two years later, the assembly created a road use tax fund, with specific allocations to FM roads.

Development of the system proceeded slowly, beginning with designation of specific roads as FM roads. An earlier bill, passed in 1939, established the concept of FM roads and caused up to 10,000 mi to be so designated. In addition, progress on grading and surfacing from 1939 to 1947 was limited somewhat by the lack of funds caused by the depression of the 1930s and by the diversion of resources caused by national needs of World War II.

As the volume of work on secondary roads increased, the need for regular liaison between the State Highway Commission and county engineers became apparent to all. Therefore, the Secondary Road Department was established on April 7, 1953. (Its current title is Office of Local Systems.) In addition to its liaison function, the new department was able to act as a conduit to provide the services of all departments of the highway commission to the counties.

A year later, the highway commission adopted the 1954 Secondary Road Plan. The plan represented an agreement with the Bureau of Public Roads whereby the counties agreed to certain procedures, standards, and general operational rules for construction on the combined FM and federal aid secondary system. As conditions have warranted, the plan has been updated. Of particular interest was the requirement to meet the current FM road standards, beginning with the standards approved in January of 1954.

AASHO design standards were used as the basis for the 1954 Iowa standards, with some modifications to fit Iowa conditions. Width of roadbed was based on traffic volume, in vehicles per day (vpd). Maximum widths of roadbed were specified, based on average daily traffic (ADT) and ranged between 22 and 30 ft. A minimum of 2-ft shoulders was required for any pavement and an additional 2 ft of roadbed width was permitted where fills in excess of 6 ft were necessary. Foreslopes were to be 2:1 to 3:1 in cut or in fills of less than 4 ft and 1.5:1 or 2:1 in fills over 4 ft. Backslopes of 2:1 were permitted, but flatter slopes needed to be justified by the need of borrow material for adequate fills or when adjacent property owners requested flatter backslopes (provided they waived all right-of-way costs). New bridges could be 16, 20, or 24 ft in width, and nonpassing sight distance was based on an eye level at 4.5 ft above the road and at design speeds lower than currently used.

Most of the high-type surfacing on these roads was placed after 1954, with the bulk of the pavement being placed in the late 1950s and 1960s (see Figure 1). By 1963, 17 percent of the FM roads had been paved with higher type surfacing, and 81 percent had been surfaced with gravel or crushed stone.

Pavement was designed for a service life of about 20 to 25 years, which has proven to be reasonably accurate. Surfaces placed during the 1950s began needing significant reconstruction in the middle-to-late 1970s and, because the bulk of the pavement was placed in the 1960s, much of the paved FM mileage faced significant reconstruction or resurfacing needs in the late 1980s and will continue into the 1990s.

Since 1954, design guides have been upgraded several times, making a large portion of these roads nonconforming when compared to current design criteria. It is expected that significant improvements to these roads will be necessary as they are reconstructed or rehabilitated, to conform to current design guides. The last update was in 1985, based on the 1984

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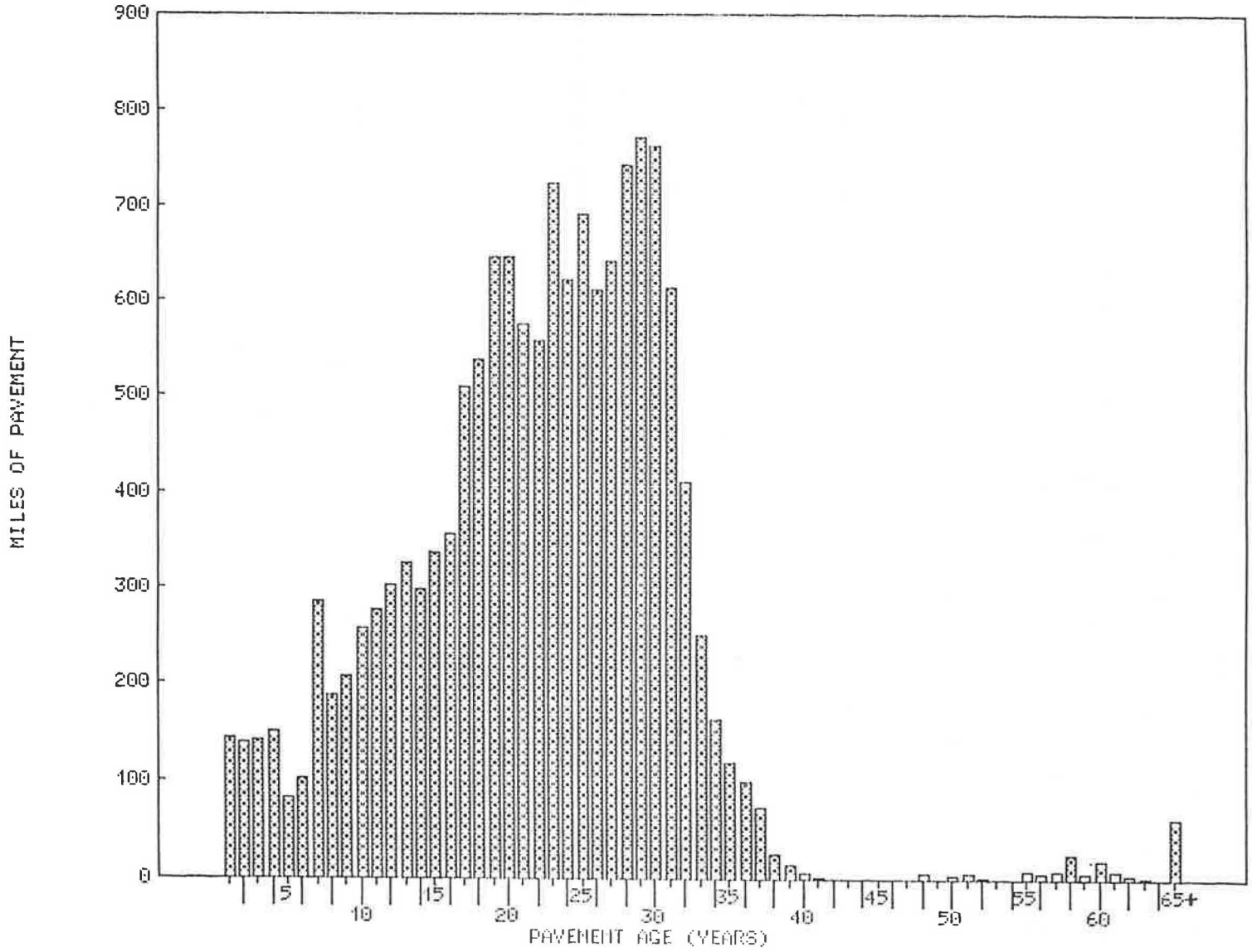

FIGURE 1 Age of secondary paved roads as of 1988.

AASHTO Green Book, "*A Policy on the Geometric Design of Highways and Streets*" (*1*). In many instances, the cost of upgrading a given road to current criteria may be quite high and not be cost-effective.

In addition to the higher costs, there was concern on the part of county engineers and boards of supervisors about the possibility of an accident's occurring on a road that had undergone some improvements without meeting current design guides and the potential for tort claims filed against the county citing failure to adhere to current design criteria (*1*). (In 1965, the Iowa legislature passed the Tort Claims Act, which is now part of Chapter 25A of the Code of Iowa. It waived governmental immunity for negligent acts on the part of the state. In 1967, the legislature passed the Tort Liability of Government Subdivisions Act, Chapter 613A, which waived immunity for any government subdivision.)

The concern seems quite valid, because recent judgments in tort liability cases have often resulted in some large awards and public agencies have been particularly hard hit. The issue of adherence to design criteria often arises during the legal proceedings. Although Iowa has an exception in the code on resurfacing projects, it may not apply to other resurfacing, restoration, or rehabilitation (3R) projects and does not apply to reconstruction projects.

Yet, it seems logical to consider the option of not upgrading entirely to current design criteria in some instances if the cost is great and corresponding improvement in safety is questionable. As a result, requests for variance from current design guides were made to the Local Systems Office, that were reviewed and variances often granted, but without the use of any specific guidelines for the decisions. This has occurred not only on projects involving 3R but also for projects involving reconstructions. [Projects falling into the rehabilitation and restoration categories are reviewed against current design minimum AASHTO guides, whereas resurfacing projects are reviewed against separate guidelines (*2*)].

IOWA DESIGN EXCEPTION PROCESS

In 1985 and 1986, an engineer in the Local Systems Office of the Iowa Department of Transportation formulated a rational method for reviewing county requests for variances from current design criteria. This method, referred to as the Iowa Design Exception Process (IDEP), provides for the use of a simplistic benefit-cost analysis of county road improvements. It is used as a rational method of granting or denying an exception to the current design guides. The majority of these exceptions are on 3R projects with geometric features that require significant and expensive upgrading to meet current design guides.

The IDEP guidelines developed were in response to the FHWA Technical Advisory T–5040.21, addressing the issue of "Geometric Design Criteria for NonFreeway RRR Projects" and to the desire for a cost-effective approach to these 3R projects using the subject road's accident history.

Technical Advisory T–5040.21 addressed such issues as practicality and need for specific improvements—especially pavement condition, traffic volumes and characteristics, traffic safety and safety needs, and economic considerations.

The IDEP document went through the normal internal and external review and approval process and was forwarded to FHWA Office of Engineering and Operations in Washington, D.C., from the FHWA Region 7 office. IDEP was described in an FHWA memo as being "well thought out and encompasses a reasonable approach towards assuring safety enhancement on 3R projects" by FHWA and "an example of what can be done to assure proper mitigation of design exceptions." IDEP is currently in use as a means of evaluating requests for exceptions to current design guides for reconstruction projects as well as 3R projects. (Most of the use is for 3R projects.) The review and justification procedure is described briefly in the next section.

Justification Procedure

A decision as to whether to plan improvements for a given road is based on a number of factors, including pavement condition, accident experience, potential impacts on the surrounding land and development, and economic benefit to the existing highway system. Few of the FM roads in Iowa experience traffic flows that would cause the Level-of-Service (LOS) to fall below LOS B, though traffic might be the basis for choosing reconstruction over 3R. Whatever the reason or reasons for the decision to make a given improvement, the county may find that a proposed improvement may not meet one or more of the current design guides. This would not be an exceptional case, because current design guides have been upgraded in several areas from the 1954 version. Most difficult to upgrade would be sight distance, road width (including shoulders), ditch cross section, and bridge widths.

However, some improvements requiring upgrading to meet current design criteria can be economically justified on the basis of expected benefits accrued from an accident severity reduction compared to the costs of the improvement. But other improvements that include provisions to upgrade to current design guides may not be considered cost-effective. What was needed was a rational, objective procedure that could logically be used to justify an exception. Instructional Memorandum (IM) 3.216 (*3*) to County Engineers has provided the mechanism for the justification of exceptions.

Therefore, since 1986, should a county find itself in a position in which a proposed improvement does not meet one of the current design guides, the following procedure is used, as outlined in IM 3.216 (*3*).

1. The first step is to acquire accident data for the road. Accident data are available through the Iowa DOT, using the Accident Location and Analysis System (ALAS). ALAS can retrieve fatal, injury, and property-damage-only accident data for all road systems in the state. Data are then analyzed to determine locations for all accidents and related contributing circumstances. Of special interest would be any roadway environment contributing circumstances, but other accidents should be analyzed to determine whether improvements in roadway environment (flatter slopes, additional signs) could have lessened the severity of the accident.
2. The second step is to prepare an estimate of the cost to bring the deficiency or deficiencies up to current guidelines.

Estimates can be based on average conditions and are not expected to be precise.

3. Third, calculate the benefit-to-cost ratio, using the accident data, cost estimates, and the prescribed benefit-cost determination process outlined in IM 3.216.

The prescribed benefit-cost determination process uses cost figures based on the estimated cost of the improvement and total accident benefit. Total accident benefit, the benefit term in the ratio, is a composite figure, based on the road's accident history since inception of ALAS (1979). (A sample worksheet is shown in Figure 2.) Briefly, the benefit is computed as follows:

1. Total loss from accidents is computed for the project, based on Iowa-developed costs of fatalities and injuries and actual values for the property damages associated with the accidents for a specific number of years.
2. Cost per accident is computed, using the total loss computed previously and dividing by the number of accidents.
3. The accident rate is computed and expressed as the number of accidents per hundred million vehicle-miles (HMVM) for roadway section and accidents per million entering vehicles (MEV) for spot locations, based on road accident history.
4. Total accident loss is then computed as the product of cost per accident, accident rate, and estimated traffic volume over the estimated service life (ESL) of the proposed improvement.
5. The total accident benefit represents an estimate of the savings expected—an estimate of the reduction in accidents or accident severity, because a given design feature is brought up to current design guidelines. Estimated accident or accident severity reduction factors for the various types of improvements and estimated service life values are provided as part of IM 3.216 (*3*).

Most of the percent reduction factors used in computing the total accident benefit are either taken from *Accident Reduction Levels Which May Be Attainable From Various Safety Improvements*, published by the federal Office of Highway Safety in 1982 or Iowa's own update. Iowa's update is based on Iowa's experience along with the University of Kentucky's Research Report UKTRP-85-6, *Development of Accident Reduction Factors*, and used to keep reduction percentages current. Changes are reviewed by engineers from the Office of Local Systems, FHWA division office, and the Bureau of Transportation Safety (BTS).

The benefit-cost ratio is then computed, using the total accident benefit (computed as described) and the estimated cost of the improvement. This figure is used in reviewing the application for a design exception.

If the benefit-cost ratio is high (>1.00), then obviously the improvement, designed to meet current design guides, is feasible. Conversely, if the ratio is low (<1.00), then it is not cost-effective. However, this is not the usual form for using this economic evaluation technique. Under normal circumstances, a discount rate is used as part of the evaluation. But although the time line for costs can be predicted, it would be impractical at the local level to try to estimate the time line for accidents. Therefore, no discounting is taken into account. (Perhaps using the term "benefits per dollar of outlay" would be better for IDEP, because this method of comparison does not use discounting.)

Therefore, review of the application uses three different ranges of values for the benefit-cost ratio:

1. If the ratio exceeds 1.20, then the improvement is probably cost-effective and should be completed in conformance with current design guides.
2. If the ratio is less than 0.80, then the improvement is probably not cost-effective if current design guides are followed. Therefore, approval of the exception is likely providing steps are taken to mitigate the situation—to provide some means to make the location safer, short of attempting to meet current design guides. An example would be the use of some positive guidance techniques, in the form of signs or pavement markings (such as edgelines, advisory speed plates, and chevrons).
3. Should the ratio fall between 0.80 and 1.20, then a more in-depth review is in order. A more detailed study of the accident data might reveal that most had occurred at spot locations along the project, suggesting that these spots be upgraded, while the rest of the project not be required to meet current design guides. IM 3.216 (*3*) provides some guidance on this review, which would be used to provide additional input to determine whether the improvement would be considered to be cost-effective.

The range of 0.80 to 1.20 was established after consultation between engineers from the Office of Local Systems, FHWA division office, and the BTS. There was a desire for a more careful review when the benefit-cost ratio was within 20 percent of the break-even point (ratio = 1.00). Therefore, additional consideration is given to (a) accident rate compared to statewide average, (b) type of accidents versus type of improvement, (c) severity of accidents, (d) cost of improvement versus project costs without improvement, (e) environmental and social effects of improvement, and (f) other alternatives to the improvement.

The County Engineer provides the data on forms developed by the Office of Local Systems for this purpose plus a cover letter requesting the exception. The letter would include the county justification (reasons) and any steps proposed to mitigate the exceptions covered by the request.

Upon receipt of a request, Office of Local Systems will review data provided and either approve or disapprove the exception. For projects not involving any federal funds, written approval from the Office of Local Systems is sufficient. But for projects that involve federal funds, requests that have been approved are forwarded to FHWA, along with Iowa DOT's recommendation.

Project Experience

The first full year of IDEP was 1987, with a number of requests for exceptions in 1987, 1988, and 1989. Although there was some resistance to the process initially from the county engineers, it is now well accepted. It has been used both for reconstruction and 3R projects, but the majority of the requests have been for 3R projects.

County_____

BENEFIT/COST DETERMINATION
(Rural Roadway Section)

Project No._____ Date_____

Location_____ Prepared by_____

Length_____miles Current ADT_____

ACCIDENT DATA: From _____ (date) to _____ (date), Total # _____years

Fatal Accidents _____ # Fatalities _____ x $435,000 = $ _____

Injury Accidents _____ # Injuries _____ x $ 15,000 = $ _____

Property Damage Acc. _____ Actual Prop. Dam. (Total) = $ _____
(Use $900/Acc. if none given)

(1) Total # Acc. _____ (2) Total Loss = $ _____

(3) Cost/Acc. = $\frac{(2)}{(1)}$ = Total Loss/Total # Acc. = $ _____/accident

(4) Acc. Rate = $\frac{\text{Total \# Acc. x 100,000,000}}{\text{ADT x Length x years x 365}}$ = _____ Acc./HMVM

IMPROVEMENT BEING CONSIDERED:

Description of Improvement:

(5) Estimated Cost $_____ (Thousand)

Estimated Service Life (E.S.L.) ____years

Estimated Percent Reduction in accident/severity _____percent

B/C ANALYSIS:

(6) Estimated Traffic Volume =

$$\text{ADT} \times \frac{1 + (1.02)^{\text{E.S.L.}}}{2} \times \text{E.S.L.} \times \text{Length} \times 0.00000365 = _____ \text{ HMVM}$$

(7) Total Accident Loss = (3) x (4) x (6) =
Cost/ Acc. x Acc. Rate x Est. Traf. Vol. = _____ (thousand)

(8) Total Acc. Benefit = (7) x % reduction =
Tot. Acc. Loss x Est. % Acc. Reduction = _____ (thousand)

$$\text{Benefit/Cost Ratio} = \frac{(8)}{(5)} = \frac{\text{Tot. Acc. Benefit}}{\text{Est. Cost Imp.}} = _____$$

FIGURE 2 Sample worksheet for IDEP.

In order to better understand how IDEP has been used, all projects during the period 1987 through 1989 were analyzed for a sample of 12 of the 99 counties in the state. Variation in topography in the state is well represented in the sample, and some urban counties were selected, along with predominately rural counties.

There were a total of 134 FM or federal aid projects from the 12 counties during the 3 years, or about 4 projects per county per year. A total of 82 (61 percent) of the projects were classified as either new or reconstruction, whereas the remaining 52 (39 percent) fit the guidelines of IM 3.214 (*2*) for 3R projects. Of the 82 new projects, a total of 61 were bridge replacement projects only and not needing IDEP review. Of the remaining 21 projects that involved grading or paving, design exceptions for vertical curves were granted for 6 projects.

An examination of the 52 3R projects revealed that about 30 percent, or a total of 16, were granted design exceptions. The 16 design exceptions granted break down as follows:

Exception Category	*Frequency*
Vertical curves	9
Horizontal curves	1
Foreslopes	2
Shoulder widths	1
Bridge widths or guardrail variance	3

Case Study

One of the 12 counties was selected for a more complete study of its design exception experience. During the 3 years studied, five projects received approval for exceptions to existing design guides. Bids were received on three projects in 1987, one in 1988, and one in 1989. Some details about the projects are as follows:

Project	*Type*	*Length (mi)*	*ADT (vpd)*	*Deficiency*	*B/C Ratio*
1	resurfacing	2.0	410	vertical curve	0.66
2	resurfacing	3.0	160	vertical curves, foreslopes	0.39
3	resurfacing	2.0	1,480	foreslopes	0.19
4	resurfacing	2.9	301	vertical curves	0.60
5	resurfacing	6.0	610	vertical curves, shoulders, and foreslopes	0.68

A comparison of the exceptions granted for the sample of 12 counties and the county selected for the case study shows strong similarities, with vertical curves being the predominate reason for the request for variance. All of the exceptions were granted for resurfacing projects, which is not surprising, in view of the pavement ages throughout the state, as shown in Figure 1. Traffic volumes on the roads varied considerably, suggesting that it would be inappropriate to surmise that exceptions would only be granted on the lightly traveled roads. Benefit-cost ratios for the projects were all sufficiently below the ratio of 0.8 set by IM 3.216 (*3*) for "improvements probably not cost-effective at this time."

Steps taken to mitigate the vertical curve exceptions included painting of edge lines and marking and resigning the no-passing zones for the current height-of-eye requirements. Painted edgelines were also used where shoulder width failed to meet current design guides.

A close examination of the completed applications for variance showed an interesting facet. The county engineer had obviously carefully examined available information about accidents on each project. In several instances, it was noted that given accidents did not apply to the design condition covered by the application. They included accidents at a narrow bridge site that has subsequently been replaced with a wider bridge, intersection accidents having nothing to do with the variance requested, and accidents involving animals in the roadway. In each instance, the logical step was taken to remove the accidents from consideration and, in each case, the removal aided in making the case for approval of the requested exception.

Acceptance of IDEP

When IDEP was first introduced in 1986, there was some resistance on the part of some county engineers. It would involve more forms to complete and more work for the engineer and would not be a better system than the one currently in use.

However, engineers from the Office of Local Systems approached the problem of acceptance of IDEP in positive ways. They conducted familiarization sessions with groups of county engineers to familiarize them with the process and how to complete an application. In some instances, they assisted the county engineer in completing an application.

The county engineers found that it was easier to justify a request for variance than anticipated. Reviews were easier and approvals seemed easier to get, because guidelines were clearly stated. County engineers found that they became more aware of high-accident locations on roads in their jurisdiction. Communications improved between some engineers and local law enforcement agencies, as the agencies were made aware of the need for good and timely accident information. (Sometimes the engineer would not be aware of an accident until ALAS records were requested and received.)

In its 4 years of use, IDEP has gained acceptance from most of the county engineers. In one instance, an engineer who was originally firmly against it is now one of its strongest supporters.

CONCLUSION

It is apparent from examination of projects in the 12 counties sampled that IDEP is working. Exceptions are not granted or requested in a majority of projects, because most of the projects meet current design criteria or the improvements upgraded to meet current design guides can be shown to be cost-effective. The high approval rate of requests for variance suggests that county engineers are not submitting requests when the improvements are demonstrated to be cost-effective.

High-accident locations are being identified and corrected, whereas less critical locations receive treatment to make them safer, such as better signs and additional pavement marking.

As might be expected, the exception most often encountered is vertical alignment. The change from 4.5-ft height-of-eye to 3.5 ft would require considerable changes in gradient in many locations, with concomitant high costs, yet safe stopping sight distance does not appear to be a significant accident factor on secondary roads.

It is, perhaps, too early to gauge its effect on tort liability claims and judgments against local jurisdictions, but to date there have been none filed involving projects covered by IDEP variances granted. It may take a few more years of use of IDEP to really judge its effectiveness in protecting county administrators from tort liability claims based on failing to meet current design guides, but at least initial tort liability experience (or lack thereof) has been good.

Iowa has had enough experience with IDEP to recommend the concepts demonstrated by IDEP to other jurisdictions seeking ways to make decisions on exceptions less subjective. It seems to be a more defensible approach than that used previously. In addition, the engineer-applicant is better informed regarding the basis for the request for variance. It also provides the decision makers with some good comparative numbers on which to base a decision, which makes the job easier to do.

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2. Instructional Memorandum 3.214. Office of Local Systems, Iowa Department of Transportation, Ames, Oct. 15, 1985.
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Simple Technical Model for the Maintenance of Low-Volume Roads in Austria

JOHANN LITZKA AND WOLFGANG HASLEHNER

A new model for the systematic maintenance of rural roads in Austria is described which has been elaborated in the course of a research work for the Austrian Ministry of Agriculture and Forestry. The main part of the work deals with the development of a technical maintenance system consisting of the following individual steps: (a) Inventory and documentation of the existing roads and their conditions (data base) as a basis for the exact characterization and assessment of the individual roads and the assignment of further data. (b) General data collection that primarily aims at the recording of the condition of the network. In this, the main target is not the exact recording of the conditions of the roads with view to individual damage features, but the distinction between roads or sections in good conditions and those for which a further, detailed investigation is demanded. (c) Detailed investigation of the sections for which rehabilitation or reconstruction measures are necessary. The detailed investigation aims at the development of the maintenance project, including the selection of the technically appropriate and the economically most efficient variant of the rehabilitation or reconstruction measures which might be applied. (d) Priority rating of individual measures within the considered network. The indicators that are necessary for the rating are the condition of the road, number of users, traffic category, traffic load, and the length of the section that needs rehabilitation or reconstruction.

The rural road network consists of all roads that are neither federal nor provincial roads and that serve the purpose of rendering possible an areal access to the rural area. This low-volume road network consists of all municipal roads, farm roads, and forest roads. Its total length in Austria amounts to about 150 000 km.

As the responsibility for construction and maintenance with view to this road network is split up between private persons and public authorities there is no exact documentation as to its length and condition. In practice up to now, maintenance measures were only carried out randomly.

On behalf of the Austrian Ministry of Agriculture and Forestry, therefore, a proposal for a maintenance system for rural roads has been developed at the Institut for Transportation Engineering of the Vienna University of Bodenkultur (*1*). The system was elaborated in steady contact with the representatives of the appropriate province authorities, which ensures that practical requirements have been incorporated as well.

Thus, a system can be offered that is simple and easy to handle but is in itself methodologically and technically consistent. It is based on already existing solutions for the primary as well as for the secondary road networks (*2–12*) and takes into consideration the possibilities and needs of the agencies in charge of the construction or the maintenance of rural roads in Austria.

The testing of the system in practice has been finished. At this time, the system is adapted to be installed in a personal computer so that the processing of the data can be handled easily.

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GENERAL FOUNDATIONS

Definitions

In order to differentiate between the various activities in the maintenance of roads, the following terms and concepts are used in Austria.

Operational Maintenance

Operational maintenance includes winter maintenance (clearing and spreading) the clearing of the carriageway and parking places of contaminations, the maintaining of the planting (cutting of plant growth and maintenance of the necessary clearance) and eventually also the examination and restoration of road signs and road markings. Measures of this kind can hardly or only to a small degree be planned in advance. They are either carried out on the basis of a condition recording according to needs arising or in regular fixed intervals.

Constructional Maintenance

Constructional maintenance includes all measures that serve the maintenance of the road and the roadside ancillaries.

Constructional maintenance can be divided into three kinds of maintenance activities. Figure 1 shows these three kinds of maintenance activities within constructional maintenance.

Routine Maintenance Routine maintenance includes all measures that are unavoidably and immediately necessary to preserve the structural quality of a pavement and its level of service. Generally, these are measures that are carried out continuously and instantly, like, for example, small repairs.

Constructional Maintenance		
Routine Maintenance	Rehabilitation, Restoration	Reconstruction, reconstructional alterations

FIGURE 1 Activities in constructional maintenance.

They must be carried out as soon as possible after the deficiencies have been recognized to repair the damage and to avoid the development of secondary damage. Long-lasting effects, however, cannot be expected as a consequence of routine maintenance measures. Measures of this kind are neither included in a long-range maintenance program nor in a rating of priorities, but are carried out periodically according to needs arising and as soon as possible.

Rehabilitation Rehabilitation activities include all extensive measures that are applied to preserve the Level of Service or to improve the structural qualities. In practice, these measures are applied to surface layers and wearing courses in longer road sections and at least to the full width of traffic lanes, such as, for example, the placing of a wearing course. Rehabilitation measures may already effect an improvement of the serviceability of the road, especially in cases for which the placing of additional bases or wearing courses on the existing pavement results in an additional strengthening effect. Rehabilitation measures will be applied at periodical intervals even if the necessary measures of a routine maintenance are regularly carried out. Regular and proper routine maintenance activities will, however, considerably extend the interval between rehabilitation measures.

Reconstruction and Reconstructional Alteration Reconstruction measures include so far-reaching measures that the final result is equivalent to a new pavement, which means that no rehabilitation measures caused by structural damage are necessary within the usual life cycle.

A reconstructional alteration is a reconstruction with simultaneous alterations of alignment elements or the width of the cross section of the road. Reconstruction also includes the reconstruction of drainage channels and other drainage facilities. The boundaries between these three kinds of maintenance activities are fluid.

From the practical point of view, however, an exact differentiation does not seem necessary, so that this basic framework may primarily serve for a rough assignment of individual maintenance activities to one of the three kinds mentioned.

Special Circumstances in the Maintenance of the Rural Road Network

An essential difference to the maintenance of federal and provincial roads is the fact that in rural roads the persons or institutions who are responsible for the maintenance are almost always different from those who carry out the maintenance works. With the exceptions of municipalities who run their own maintenance depot or special cases of maintenance cooperatives, orders for maintenance works are either placed with firms or they are carried out by the departments responsible for the construction of rural roads themselves either for the municipalities or for the group of persons whose interest is the maintenance of the road.

Even though in most of the cases the second of the mentioned possibilities works well in practice, there is actually never a basis for an obligatory cooperation between the institution or department providing the maintenance funds (financier) and the technical agencies (the technical advisers or the staff carrying out the maintenance works).

The institutionalization of such a cooperation as a basic precondition for a systematic maintenance of rural roads might above all be achieved through the provision of appropriate financial funds [cf. *Fund for the Maintenance of Rural Roads of the Province of Salzburg* (*13*)].

Contrary to that, the application of methodological maintenance systems is much easier in the federal and provincial road network. In these, road construction departments or road maintenance depots are responsible for the maintenance of the road network within their competence on behalf of the authority providing the maintenance funds (federal or provincial government). Within the bounds of the allocated funds, they are directly responsible for decisions concerning the time and priority of maintenance measures as well as for the realization of the maintenance works (whether they carry them out themselves or entrust a firm). The whole management of the maintenance is thus undivided.

The previous considerations are only relevant as a background for the application of the system and not for the system itself. A basic consideration for the development of the model was that it should be applicable to all cases that might occur in connection with the maintenance of rural roads. It is intended to be applicable for individual municipalities as well as for a whole province. It has to be noted, however, that the application of objective decision models is the more useful the larger the network is on which they are based as the advantages of an objective assessment and listing of measures according to priorities can only then be fully effective.

To take account of the given situation, the proposed system has to imply the possibility of a flexible application to achieve general acceptance. That means that it has to be considered that

- The possible users (municipalities, construction departments, and provinces) have up to now dealt with problems of systematic maintenance in a quite different way and degree; and
- The roads making up a network may often reveal quite essential differences as to their state. This condition applies especially to cases for which the model shall also be used for

the maintenance of roads that were not built according to the respective state-of-the-art.

As in other cases in which a new system is installed, the introduction of the described maintenance model in the initial state will naturally lead to organizational difficulties as well as to problems in the acquisition of the necessary basic data, the general inventory of the network, and the like. Of course, it will be necessary to schedule a longer period for these time-taking preliminary and initial works.

As soon as the basic inventory and the preliminary works have been made for the whole network, the acquisition of the data will require much less expense. These routine works mainly imply the updating of information already documented or the extension of the information basis for decisions taking into account economic criteria (e.g., current price increases).

Strategy on Which the Maintenance Is Based

Basically, there are two extreme strategies for the maintenance of the road network. The first strategy does without measures of routine maintenance and regular rehabilitation. When the condition of the road falls below a level that is acceptable, the road will be reconstructed.

An alternative to that is the strategy that applies measures for the best possible routine maintenance connected with periodic rehabilitation measures.

Figure 2 shows a schematic representation of the two different strategies and the development of performance curves (development of deterioration) resulting from their applications.

It can clearly be seen that Strategy 2 results in a substantially higher serviceability over the whole time span and thus in relatively low decreases of the pavement condition and traffic safety.

If Strategy 1 is applied, however, the condition of the pavement remains for a long time below its initial condition. On the basis of technical and economical considerations, it can be concluded that the strategy that includes routine maintenance is the better one. Routine maintenance in time reduces the danger of the development of secondary damage and thus prevents the progressive development of damage. Furthermore, it is normally easier to invest smaller sums continuously, whereas the funds for a reconstruction at the proper time are often not available.

The presented maintenance model is based on the assumption that the strategy involving routine maintenance measures is applied. Routine maintenance measures are therefore not an issue for detailed considerations and priority ratings but are normally carried out periodically. But this is only true for cases in which sufficient financial means for routine maintenance works are provided. If these means are not available, a deterioration of the average level of the network must be expected.

MODEL FOR TECHNICAL DECISIONS

General Remarks

The proposed model for technical decisions on the maintenance of rural roads will be discussed in detail in the following. In accordance with the considerations and targets presented previously, it must be applicable to rural road networks consisting of any possible pavements. This is important, especially in cases where the decision-taking institution has practically no information on the existing network. So, the model is divided into the individual steps of

- Inventory, determination of the boundaries of the network;
- General recording of the condition in order to quickly get a survey over the kinds of pavements and the conditions of the pavements of the individual roads;
- Detailed assessment of the sections for which maintenance or rehabilitation measures are necessary, with additional investigations;
- Decisions on measures to be applied in the respective section (maintenance project). As the solution resulting from this procedure refers to a certain project, this step is called decision on project level.

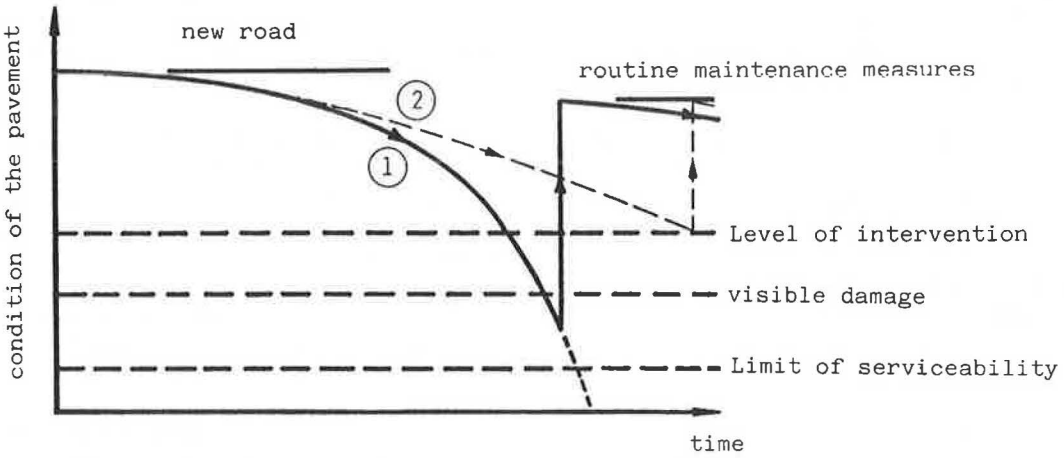

FIGURE 2 Performance curves in different strategies.

If sufficient information on the structure of the network is already available, this step of the inventory may be omitted. Also, the general recording of the condition can be dispensed with if there are relevant actualized recordings. In this case, it is possible in the presented model to start with the step of detailed investigations. In routine application—after the ending of the initial state—this will be the normal case.

The result of the technical considerations, the definite proposal for a measure to be taken in a certain section must then be subjected to an assessment of its urgency on the level of the whole network.

All these detailed considerations refer to rehabilitation or reconstruction measures. As for routine maintenance measures, it is assumed that they will be carried out periodically or shortly after it has been recognized that they are necessary.

Individual Steps of the Decision Model

Inventory

The precondition for all considerations concerning the systematic planning of maintenance measures is the recording of the road network to be included in the planning procedure and the knowledge of the condition of the individual roads or road sections.

If there are no recordings of the existing road network in the initial state, first of all a recording of all the roads of the network has to be carried out in which the names; road numbers (code numbers); positions; constructional data (length, width, year of construction); significance within the network; traffic loads; and the like of the individual roads are registered. This information forms the basis for all following considerations concerning the individual roads.

At the same time, the respective roads are divided into sections for which the characteristics remain the same. Thus, for example, a larger branched road must be subdivided at least into main road and access roads. In practice, the inventory of the network is in most cases carried out simultaneously with the general recording of the conditions of the individual roads. It is helpful to document all road data, traffic data, and the like and all successively collected condition data in a data base.

Assessment of the Condition

If it is again assumed that there is no information on the condition of individual roads in the initial state, having a survey as quickly as possible is necessary first of all.

For the recording and evaluation of the conditions of roads, it is therefore proposed to follow the procedure shown in Figure 3.

General Assessment The primary target in the first step of the recording of the state is first of all the recording of the condition of the network. This step of the recording does not aim at an exact registration of the condition of the pavement with view to individual damage characteristics. It shall rather render possible a differentiation between roads or sections in

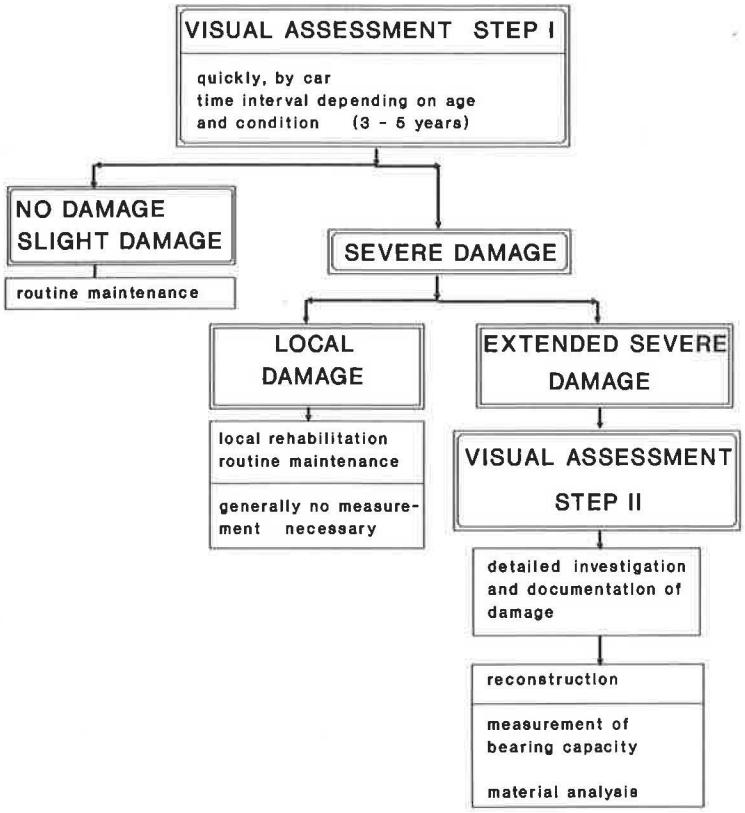

FIGURE 3 Visual assessment.

good condition and others that require further detailed investigations.

It is useful to collect the general data by driving along the roads by car. Only in this way is it possible to master the recording of the network that may be extensive within an acceptable time. The results of the data collection are taken down in a form that has been drawn up for this step. The given situation (with view to the demand of time and personnel as well as the required assessments) has been taken into account in the extent of the data to be recorded. The positioning and determination of the length of the sections is made by means of the hectometer counter within the car.

In the course of the data recording, it may prove necessary to further differentiate the division that has been made in the recording of the network into sections, if the kind of pavement or its condition changes. Only these subsections that are homogeneous in their essential characteristics are the basis for all further considerations.

For the practical realization of this recording from within a car, it seems necessary to use a team consisting of at least two experts for maintenance issues. If the recording is carried out by a larger staff (several teams), the mutual coordination of the recording procedures of the individual teams has to be considered.

As far as possible, general recordings of the condition should be made during the spring period as damage occurs to a higher degree during this season and additionally appears clearly then.

The general recording of the condition must subsequently be repeated at regular intervals to actualize the recorded data. Dependent on the conditions and life cycles of the individual roads, the interval for this systematic actualization is determined. Contrary to older roads in worse condition, a larger interval can, for example, be chosen for roads in good conditions or for lately reconstructed roads.

A maximum interval of 3 to 5 years seems appropriate to be able to register deteriorations in time and also to assign the occurring deteriorations to a certain interval of time. Only this forms the basis for the forecast of the future condition or the development of the deterioration.

If heavy local damage occurs in between the intervals that demand immediate repair, the necessary measures must be taken as soon as the damage has been realized or reported.

In routine application (when the basic recordings have been finished and the system has been installed), the periodically carried out recording of the condition may eventually be substituted by respective reports on the measures taken in routine maintenance works.

For the documentation of the recording, a form has been developed that may be used for the step of the general recording for different pavements that may occur [unbound pavements, asphalt pavements, cement-stabilized bases, and concrete pavements (see Figure 4)].

Besides the name of the road, the road number (code number), municipality, and district, also the length, the width (cross section), and the year of the construction of the road or road section are registered. Additional data give information on the quantity of traffic and the function of the road within the road network.

To determine individual subsections, if necessary, a line is given to mark their position.

The recording of the pavement condition of the road is then carried out with a differentiation according to relevant individual damage characteristics. This differentiated recording is the basis for a distinction between different deterioration states or subsections of a road.

Assessment Mark I means that routine maintenance is sufficient.

Assessment Mark II means that routine maintenance and local repairs are required.

Assessment Mark III results in a detailed investigation.

It saves time to record the condition of the pavement while driving in one direction and to record the conditions of the roadside ancillaries and other buildings on the way back.

In order to have a better survey, it is advantageous to put down the results of the assessment also in the head line of the form. If there are subsections, there may be two or three assessment results.

On the basis of this investigation, subsections with an Assessment Mark III are sorted out. These subsections demand additional detailed investigations in a further step.

From the data on the condition and the respective assessment, the interval to the next general data collection can be estimated.

Detailed Investigation

Only those roads that show extensive and heavy damage—in other words those that got the evaluation Mark III in the general assessment and were sorted out—are subjected to a detailed investigation in the second step. In these roads, expensive rehabilitation and reconstruction measures are necessary.

Thus, the aim of the detailed investigation is on the one hand the elaboration of the maintenance project including the determination of technically appropriate variants from several alternative measures and on the other hand the providing of the information needed to evaluate the priority and urgency of these measures compared to other maintenance projects on network level. The detailed investigation is carried out by an expert on maintenance activities on foot. The necessary equipment consists of a measuring wheel for the exact registration of individual deficiencies together with a rule and a rod to determine the depth of transverse deformations or of settlements.

It is again important that the methodology is objectified before the beginning of the assessment and in regular intervals while the assessment is being carried out. The best way to do so is a joint investigation of selected road sections by several assessment teams followed by a discussion and adaptation of the respectively recorded results. In order to guarantee a uniform treatment of occurring damage, a special catalogue listing deficiencies that may occur has been elaborated.

The detailed investigation is practically carried out by means of a form that is especially adjusted to the needs of a determination of appropriate measures and an evaluation of the priority and urgency of measures. For each kind of pavement (unbound pavements or asphalt pavements, cement-stabilized bases, or concrete pavements), a separate form has been developed.

Figure 5 shows the form to be used for bituminous pavements as an example.

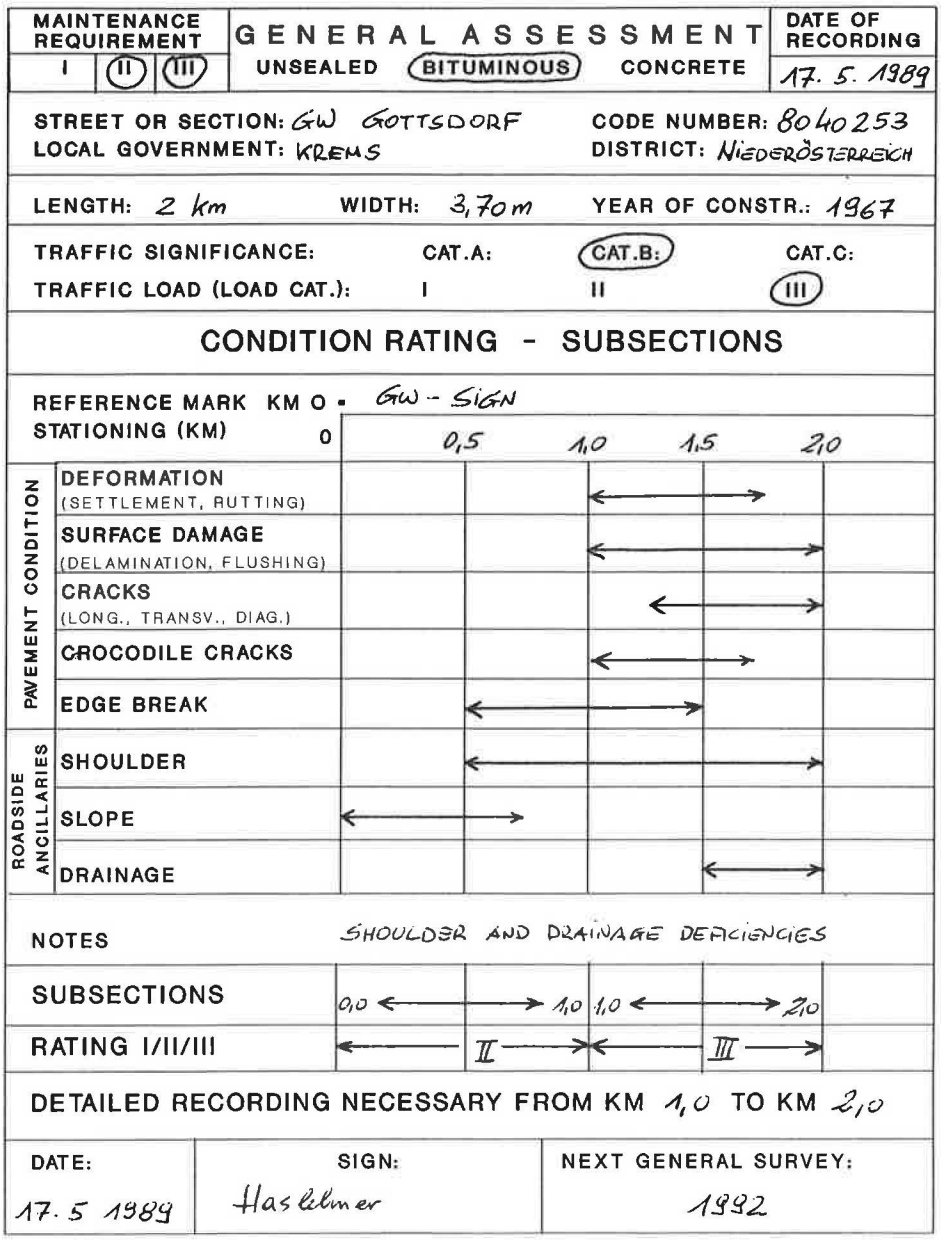

FIGURE 4 General assessment form.

General information may directly be taken over from the form recording the general condition. In addition, the number of regular users is recorded for these sections as well as particular heavy traffic that may occur (e.g., transportation of wood or tank trucks transporting milk).

The visual assessment of the pavement condition is made by recording the individual relevant damage characteristics as they are listed in the damage catalogue.

In addition, damage at drainage facilities and roadside ancillaries is documented.

The last two columns (proportion in length and extent) already deliver indicators for the successive priority rating. As to information on drainage facilities, the two values mentioned are not recorded separately, as it can be expected that deficiencies in these facilities will have a direct effect on the condition of the pavement and thus need not separately be considered in the priority rating.

Further, results of analyses of the construction material and of bearing capacity measurements are also put down in the form. At that time, bearing capacity measurements are normally carried out with the Benkelman beam under a wheel of 50 kN. The results are used on the one hand to divide the investigated section into homogenous subsections, and on the other hand for the dimensioning of the strengthening of the pavement, using, for instance, the diagram of the Asphalt Institute (*14*).

Since 1990, also two falling-weight deflectometers have been in use in Austria. Their application will in future render possible a back-calculation of E-moduli of the pavement layers and thus an analytical dimensioning of the pavement.

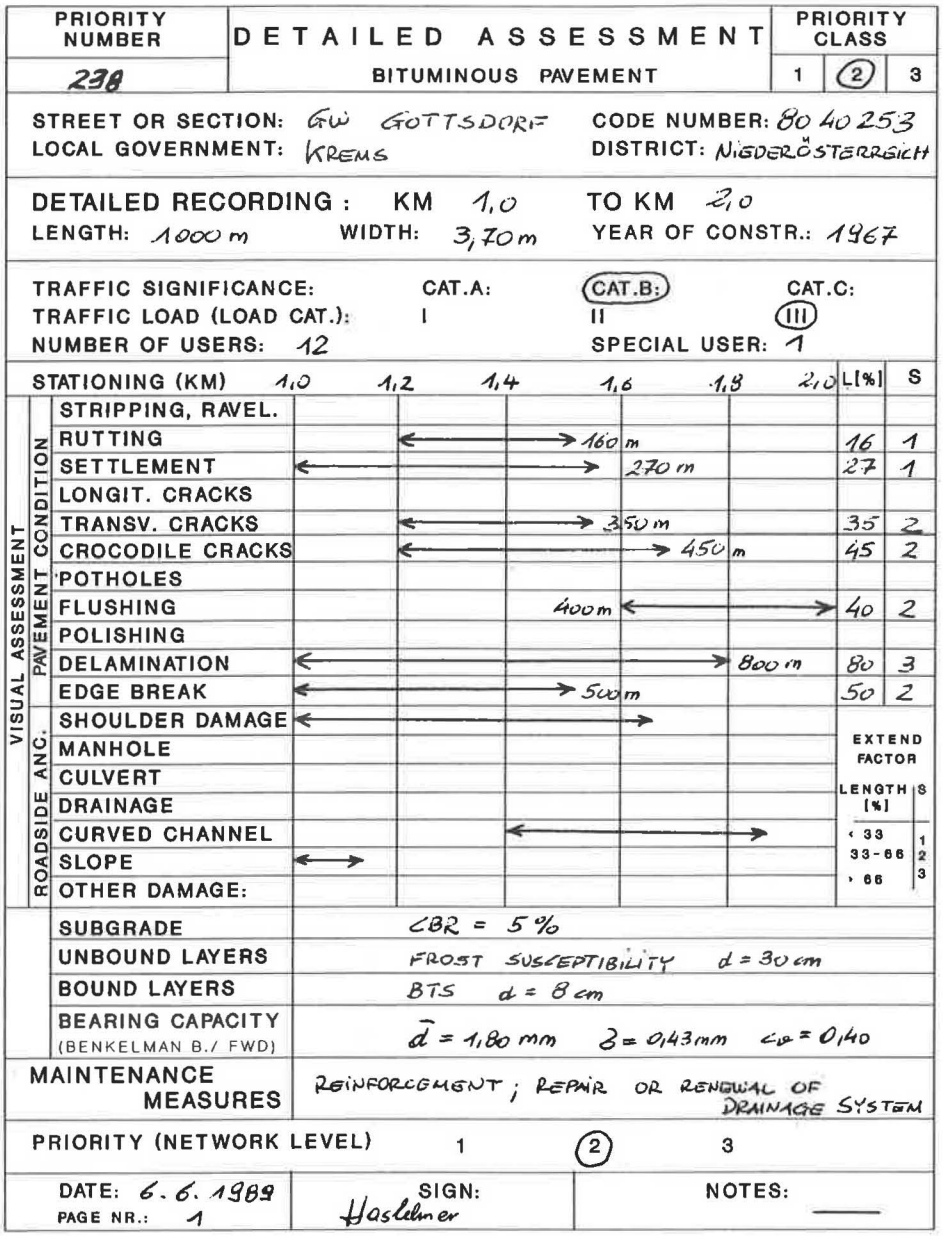

FIGURE 5 Detailed assessment form.

On the foundation of this technical basic information, the maintenance project is developed. In the course of its elaboration, several variants may be considered with respect to the given situation. The selection of a solution out of technically equivalent possibilities is based on a rating of their economic efficiencies.

The proposed measures resulting from this investigation are taken down in a form and eventually explained in a supplementary sheet.

Later on—as a result of the priority rating carried out successively—the priorities for these measures may be recorded in the head of the form.

Thus, the form for the detailed investigation provides a documentation of the condition of the section concerned, the proposed measures, as well as the urgency for the execution of these measures with a view to the whole basic network.

PRIORITY RATING

General Remarks

After a decision has been taken at project level on which of the measures for rehabilitation or reconstruction is the economically most efficient, it is necessary to rate these individual measures as to their urgency of execution with view to the whole relevant network (a decision at network level).

For this purpose, a procedure is proposed for the priority rating of rehabilitation and reconstruction measures that is especially adjusted to the conditions applying to the rural road network. This method is applicable to a network of any size, though it ought to be stressed that the appropriateness of such a priority rating increases the larger the network. A precondition for its application is a clearly defined network. This may, for example, be the whole road network that a municipality has to maintain. From the point of view of an administration providing the maintenance funds, the network might also be defined as the total of all roads for which funds have to be provided.

In the procedure for the rating to be described, only those parameters are taken into account that are either known from a detailed investigation or can be easily obtained by additional data collections.

The priority rating is made with the help of a form (shown in Figure 6). The results of the detailed investigation (damage assessment) may directly be transferred into this form. Together with the additional parameters (number of users, traffic significance and traffic load, and length of the section), it provides the basis for a simple and quick determination of the priority number.

It is further possible to assign a certain road in correspondence with its priority number to a priority class that is valid for the considered road network. In practical application, this may be advantageous as it makes the consideration of additional local criteria easier in the process of decision finding.

PRIORITY NUMBER 238 — **PRIORITY ASSESSMENT** — **PRIORITY CLASS** 1 (2) 3

STREET OR SECTION: GW GÖTTSDORF — CODE NUMBER: 80 40 253
LOCAL GOVERNMENT: KREMS — DISTRICT: NIEDERÖSTERREICH

DETAILED RECORDING: KM 1,0 TO KM 2,0

DAMAGE	EXTEND FACTOR S	IMPACT ON THE USER: IMPACT FACTOR B1	S•B1	P1 = S•B1••B2•B3	LOSS OF VALUE: IMPACT FACTOR W1	S•W1	P2 = S• W1•W2
STRIPPING, RAVEL.		1			2		
RUTTING	1	2	2		2	2	
SETTLEMENT	1	3	3		3	3	
LONGIT. CRACKS		1			2		
TRANSV. CRACKS	2	1	2	USER B2	2	4	TRAFFIC LOAD W2
CROCODILE CRACKS	2	1	2		3	6	
POTHOLES		3			3		
FLUSHING	2	1	2	2	1	2	
POLISHING		1			2		
DELAMINATION	3	2	6	•	2	6	1
EDGE BREAK	2	3	6	TRAFFIC SIGNIF. B3	2	4	
SHOULDER DAMAGE		1			3		•
FAULTING		3			2		
SPALLING		2			3		
SCALING		1		2	2		
SKID RESISTANCE		1			2		
CHANNEL		2		•	3		
		Σ S•B1	23		Σ S•W1	27	
		P1•(ΣS•B1)•B2•B3		92	P2•(ΣS•W1)•W2		27

LENGTH OF SECTION • LENGTH FACTOR L • 2

→ PRIORITY NUMBER P = L•(P1+ P2)• 2 •(92 + 27)• = 238

→ PRIORITY CLASS 2

FACTOR USER		FACT. TRAFFIC SIGNIFICANCE		FACT. TRAFFIC LOAD		FACT. LENGTH	
USER	B2	CAT.	B3	LOAD CAT.	W2	LENGTH	L
1-5	1	CAT.C	1	III	1	< 500m	1
6-20	2	CAT.B	2	II	2	500-2000m	2
> 20	3	CAT.A	3	I	3	> 2000m	3

DATE 10. 08. 1989 — DETAILED SURVEY FROM: 6. 6. 1990; Haslehner — NOTES: ———

FIGURE 6 Priority assessment form.

Therefore, a kind of uniforming effect that may occur is deliberately accepted.

The relevant parameters are discussed in detail in the following.

Parameters for the Priority Rating

Condition of the Road

First, the given condition of the road is characterized as to the kind and extent of damage occurring. Then it is evaluated with a view to the decrease of pavement condition, on the one hand, and with a view to secondary damage that is to be expected if maintenance measures are deferred, on the other hand.

Extent of Damage In the detailed investigation, every type of deficiency has been recorded in the form with the extent of its occurrence. From the length over which a certain type of damage occurs, the proportion of this damage may be calculated in percentage of the section length (last but one column in the form for detailed investigations, see Figure 5).

This proportion of length is classified and an extension factor S ranging over lengths 1 to 3 is accordingly deduced (see form, Figure 5). The extension factor is directly taken over in the form for the rating of priorities.

Impact on the User When the condition of the road is evaluated from the point of view of the road user, the decrease of the driving comfort as well as the involved safety risk is considered. According to the impacts on one of these aspects, the individual deficiencies are assigned an impact Factor B1 in which Factor 1 signifies the smallest impairment (and Factor 3 would signify the greatest). These extension factors are given in the form.

Damage at guardrails and balustrades are not taken into account as it may be assumed that this damage will be remedied as soon as possible and not only when rehabilitation measures are carried out.

Loss of Valuc (Damage Development) It is tried to evaluate indirectly to what extent the value of the pavement will decrease if the necessary rehabilitation or reconstruction measures are not realized. This evaluation is made on the damage development that is to be expected. Accordingly, the individual deficiencies are assigned an impact factor W1 signifying the different aggressivity of the individual damage types. Factor 1 signifies the least increase in damage and Factor 3 the largest one that is to be expected. This impact factor is also given in the form (Figure 6).

Number of Users

It is assumed that a measure will be the more urgent the more regular users are impaired by the bad condition of the road.

The dependence on the number of users is determined by the Factor B2 (see Figure 6).

Traffic Significance of the Roads Within the Network

It is assumed that roads of greater regional importance should have a higher Level of Service and that therefore rehabilitation or reconstruction measures should be carried out on these at an earlier time.

According to the valid standard on rural roads and ways (*15*), a division is made into three categories and each category is assigned a factor B3.

- Rural road of low traffic significance
  - —Farm roads and forest roads, B3 = 1;
  - —Access to permanent habitations, B3 = 2.
- Rural roads of greater traffic significance, B3 = 3.

Traffic Volume

It is assumed that individual damage occurrences become more severe only at a later time if the traffic volume is low, whereas a high traffic volume may eventually cause a higher degree of severity of the damage after a relatively short time. If there is a higher traffic volume also, the risk of the occurrence of secondary damage is higher.

Accordingly, roads that carry a higher traffic volume must be rehabilitated earlier.

To evaluate the traffic load, a division into load categories is made and a factor W2 is assigned to each of the load classes.

The classification of roads according to load categories is known from the elaboration of the maintenance project.

Load Category	*Equivalent Single-Axle Loads (100 kN)*	*Factor W2*
I	$\leq 5 \times 10^4$ (10 trucks/day)	3
II	$\leq 1 \times 10^4$ (2 trucks/day)	2
III	$\leq 2 \times 10^3$ (2 trucks/week)	1

Length of the Section

In the consideration of the length, the subsection which needs rehabilitation and for which a maintenance project has accordingly been developed, forms the basis. Thus, the length may be the total length of the road but it may also be only a section of it.

It is assumed that the impairment of the user but also the extent of a possible loss of value increases the longer the deficient section is. Therefore, the length factor L has an impact on both of the mentioned criteria. In the determination of the length factor, it has been roughly differentiated into three length classes (see Figure 6).

Section lengths of more than 2 km do not effect a further increase of the urgency of rehabilitation.

STATE OF IMPLEMENTATION OF THE MAINTENANCE SYSTEM

The proposed system for the maintenance of low-volume roads has been tested in practice. About 400 km of the rural road network was examined according to the described method in various Austrian provinces. For the general assessment, about

15 km per day may be recorded. For the detailed investigation, a recording performance of 3 to 5 km per day is possible.

The results are in good correspondence with the practical experience of road construction engineers. The method is merely a systematic instrument, and the decision on which maintenance measure shall be applied must still be taken by the engineer.

At present, a general assessment according to the presented concept is being started in several municipalities and provinces.

In some of the Austrian agencies responsible for the maintenance of roads, recordings of maintenance activities are made by means of index cards. Because of the extent of the rural network, this kind of documentation seems to fail to be appropriate. Furthermore, there is the disadvantage that it causes great difficulties in carrying out evaluations concerning the whole network.

For this reason, an easily handled program for a PC is developed. With this program, it will be possible to make an inventory, to document the condition, to make priority ratings, and to assess the necessary maintenance budget.

Thus, it will be possible to process the collected data quickly and to actualize them at any time desired.

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Maintenance Management Technology Training Through Case Study Approach

NEVILLE A. PARKER, ESSAM RADWAN, AND MERRON L. LATTA

A maintenance management case study approach has been developed for the International Road Federation executive conferences on road management, for senior professional engineers, and for administrators of highway agencies. A case is structured and presented as a continuing exercise that affords the conference delegates the opportunity to simulate planning, programming, budgeting, and scheduling of maintenance activities for a hypothetical network, through the practical application of real data. Delegates, working in regional groups, develop their cases, beginning with the definition of unique sets of environments that describe traffic flow, climate, roadway cross section, technology, and economic and financial characteristics. Presented with a roadway condition survey, each group then develops a first-year maintenance plan, program, and budget, imbedded in a 5-year projection designed to minimize the total cost of maintenance and vehicle operation. In the process, delegates deepen their understanding of the language and principles of road management and, at the end of the exercise, present and defend their budget requests before a simulated budget committee. Experience to date has shown that appropriately designed case studies can be effective instruments of technology transfer and training. By setting the maintenance management situation in environments defined by the delegates themselves and by providing a medium for the exchange of views and approaches to maintenance management problem-solving through group work, unique case studies can be developed that mirror reality closely and give delegates a feeling of confidence in an enhanced ability to effect meaningful change in their regions' maintenance management procedures.

The maintenance management problem is cast in the context of the interactions between surface deterioration and maintenance and vehicle operation as shown in Figure 1. Surface deterioration is a function of climate, road type, traffic, and maintenance. Road roughness, as a measure of riding surface quality, is a major link between maintenance and operating costs, because of its effect on speed and speed changes (and hence fuel consumption), as well as on vehicle wear and tear. Maintenance costs depend both on needs and strategies, whereas vehicle operation costs depend on speed and road geometry, as well as on road roughness. The cyclic interactions of surface deterioration, road roughness, and maintenance illustrate the cause-and-effect relationship between them and focuses on the strategic part of the problem. Surface deterioration is monitored or predicted, and the measure of riding surface quality determines the maintenance requirements necessary to arrest any further deterioration or to restore the surface to previous higher standards. The maintenance management problem has been fully developed in the developing country context by Gichaga and Parker (*1*). The maintenance management cycle of activities is shown in Figure 2 and described in the following paragraphs.

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Good management begins with good planning, and good planning depends on relevant, timely, and accurate data. Planning in this context means the following:

- Determining the maintenance activities to be carried out on both a fiscal year and long-term basis; and
- Estimating the resources necessary to carry out the activities (i.e. labor, equipment, and materials).

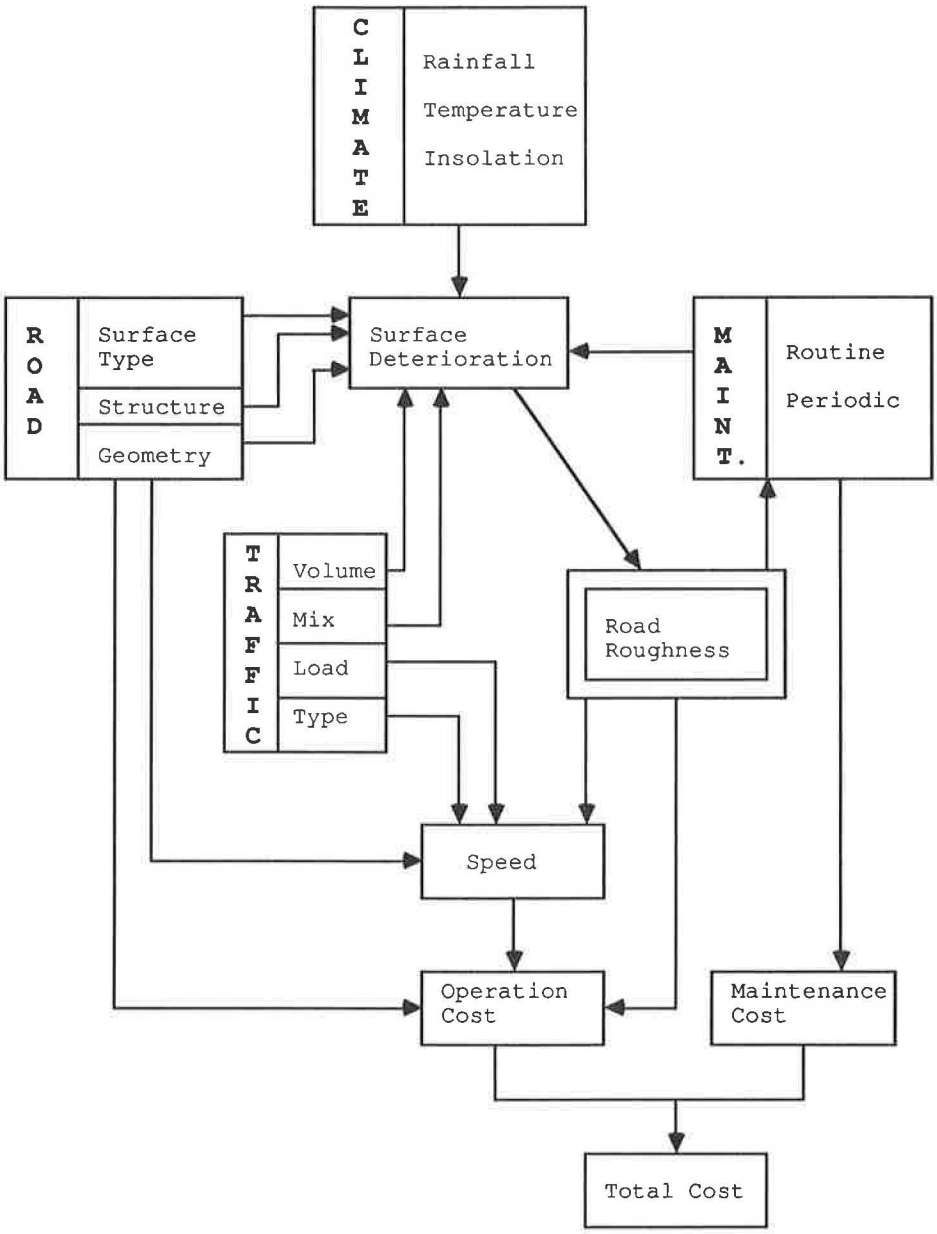
Proper planning establishes defensible budgetary requirements. However, budgeting is itself a political process, which is only initiated by the budgetary requirements. The budgetary allocations are therefore more likely to be less than the requirements. As such, therefore, it is the job of management to determine which of the planned projects should be executed to maximize the social benefits, which is expenditure budgeting, and then to revise and update the plan for subsequent years. Management is held accountable for cost-effective use of budgeted funds.

Planning is a macrolevel activity that provides a framework for measuring performance. Programming is the microlevel activity that determines the schedule of work and the detailed allocation of resources necessary to achieve the plan. It includes set up of work camps, dispatching of transport, distribution of labor and materials, scheduling of equipment, and control of finance.

To execute maintenance operations is to supervise and control them in order to ensure that the programmed activities are carried out within budget and time, and at the required standard of performance.

INTERNATIONAL ROAD FEDERATION EXECUTIVE CONFERENCE ON ROAD MANAGEMENT

The International Road Federation (IRF) executive conferences on road management are organized in cooperation with Arizona State University, and held once per year. For two



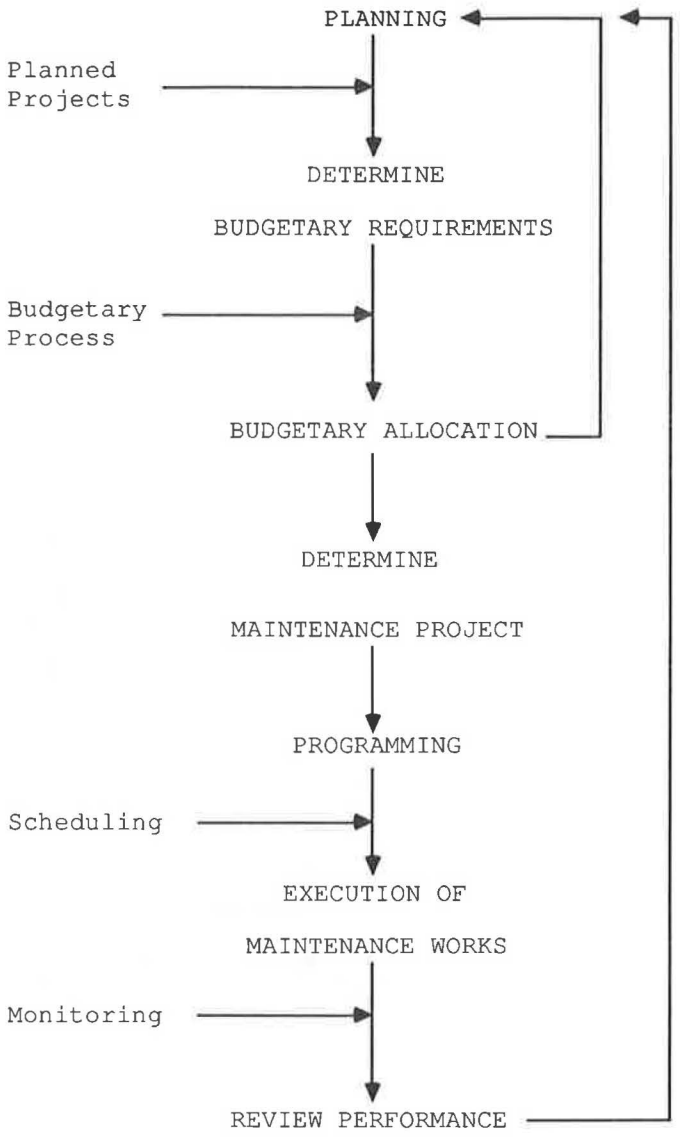
Source: Essentials of Highway Engineering (Ref. 1)

FIGURE 1 Interactions between surface deterioration and maintenance and vehicle operation costs.

weeks, delegates discuss and work together as a group and in teams, to develop solutions to the most critical problems facing road managers today. Delegates return home equipped to define their problems more precisely and make decisions more effectively. Through this educational process, the various highway agencies that the delegates represent will ultimately become more efficient.

The program features lectures, demonstrations, and structured discussion on maintenance management, pavement management, equipment management, technology transfer, and training of the work force. Site visits are made to observe each of the management operations. A maintenance management case study parallels the lectures and serves as the medium for focusing and exercising the management concepts. The typical conference is shown in Figure 3. All sessions are presented by experts from the public and private sectors and from academia. Four coordinators are in residence with the delegates to provide continuity and ensure effective technology transfer and training by providing a continuous and on-demand consultative resource. Coordinators also lecture.

The program is designed for a maximum of 30 senior professional engineers and administrators in public works and communications ministries and highway agencies, who can influence management policies and operations. Sponsoring agencies are encouraged to send multiple delegates who would work together as a team during the conference. Preference is given



Source: Essentials of Highway Engineering (ref. 1)

FIGURE 2 Maintenance management cycle.

to developing nations; highway professionals at appropriate levels in the private sector are also invited.

Delegates receive a permanent record of the conference including an abstract of each lecture, a copy of the case study manual, a copy of the reference text (*1*), as well as various monographs. In addition, each delegate is made a professional affiliate of the IRF for a period of 1 year on a complimentary basis. Professional affiliates are kept informed regularly of the latest developments in roads, and have the opportunity to subscribe to the IRF's literature on technology transfer at significant cost savings.

MAINTENANCE MANAGEMENT CASE STUDY

The case study is an exercise in decision making. It presents the delegates with a hypothetical but realistic maintenance management situation involving three major classes of urban and rural roads: namely, asphaltic premix, bituminous surface dressing, and engineered gravel. With the exception of the road geometry and condition survey, all other environmental conditions are determined by the delegates working in regional groups of three to five, though they are free to accept the default environmental conditions presented in the case handbook (*2*). After setting the case in the appropriate environments, delegates then simulate various stages of the maintenance management cycle (Figure 2), which terminates in the determination of a budget. In the process, delegates develop an enhanced appreciation for the importance of determining priorities, evaluating the consequences of alternative solutions, and making maintenance decisions that take account of both short- and long-term effects on user costs. The alternatives analysis and the exchange of views that occur within

DAY	PROGRAM
1 Sun	Arrival in Phoenix Orientation, Welcome, and Reception
2 Mon	AM : Road Deterioration Problem (World Bank) PM : Programming and Budgeting for Maintenance (World Bank)
3 Tues	AM : Principles of MMS PM : Microcomputers Software & Hardware
4 Wed	AM : Principles of MMS (cont.) PM : Case Study (Part I)
5 Thu	AM : Principles of PMS PM : Case Study (Part II)
6 Fri	AM : Principles of PMS (cont.) PM : Demos and Possible Hands-on for RTIM II and HDM III
7 Sat	AM : Case Study (Part III) PM : FREE Time
8 Sun	FREE TIME
9 Mon	AM : Equipment Management Systems PM : Visit a local agency to review their Equipment Management System
10 Tues	AM : Tutorials on Equipment Management PM : Case Study (Part IV)
11 Wed	AM : ADOT Field Trip PM : Drive to Grand Canyon
12 Thu	AM : Strategic Role of Training Case Study PM : IRF Training Tapes and Technology Transfer in Practice
13 Fri	AM : Case Study Presentations PM : Conference Conclusion
14 Sat	RETURN TO PHOENIX
15 Sun	DEPART FROM PHOENIX

FIGURE 3 Typical IRF executive conference program.

and between groups throughout the exercise are bolstered by the use of computer-aided algorithms based on the Road Transport Investment Model (RTIM2) (*3*), and the Highway Design and Maintenance Model (HDM–III) (*4*).

The Hypothetical Network

The hypothetical network of paved and unpaved roads is shown in Figure 4. It comprises four premix links, each 15 km long; four surface-dressed links, each 30 km long; and eight gravel links, each 45 km long. The paved roads connect the urban center to four major rural centers, which are in turn connected to four minor rural centers by the gravel roads. The rural earth roads shown are not included in the exercise. A detailed survey of the roadside and roadway condition of each paved link is given in the format presented in Table 1. Roughness (mm/km), looseness (mm), rut depth (mm), and gravel thickness (mm) are given for unpaved links. Additional data include classified annual daily traffic (ADT) with growth rates, roadway cross section designs, maintenance criteria, road deterioration relationships, vehicle operating relationships, unit cost items and prices, and sample outputs from RTIM2 and HDM–III to guide the organization of information. However, delegates are not constrained to accept the additional data, if in their judgment the defaults do not adequately reflect their own environmental realities. The network is divided into four districts, as shown in Figure 4.

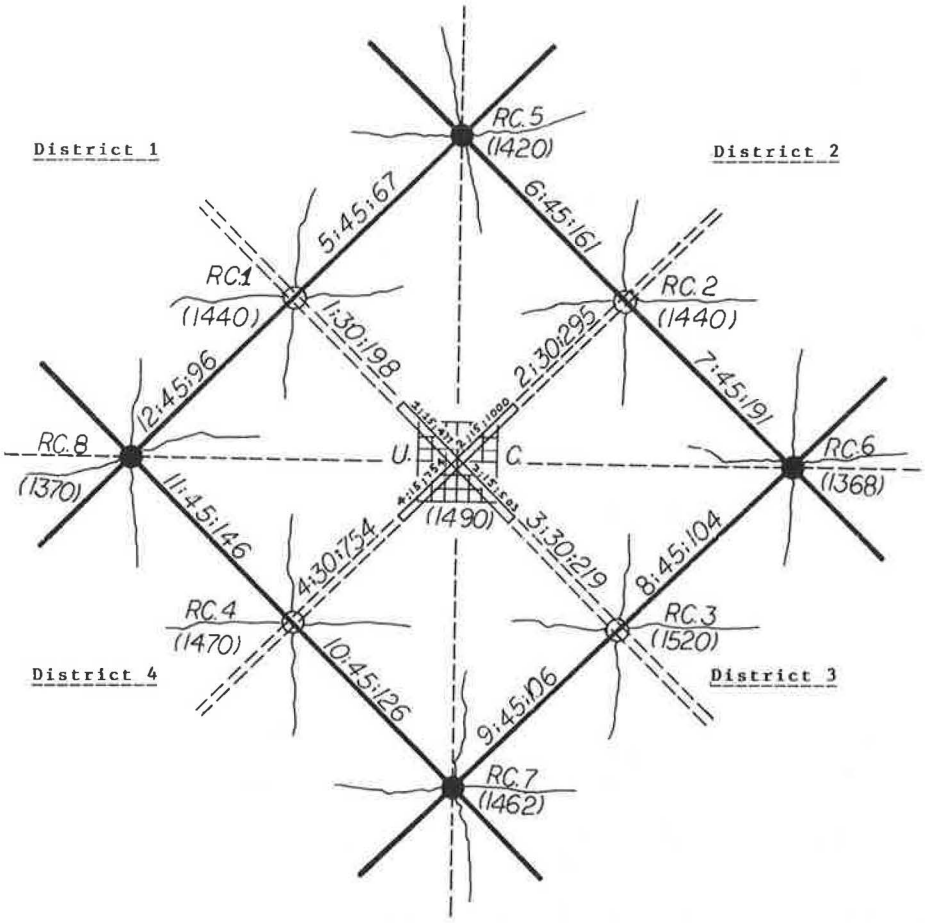


FIGURE 4 Hypothetical trunk road network.

Conduct of the Case

The case develops along the lines of the maintenance management cycle, in four exercises, as follows:

- Exercise 1: Setting environments and initialization.
- Exercise 2: Determination of maintenance requirements.
- Exercise 3: Estimation of resource and budget requirements.
- Exercise 4: Balancing maintenance requirements with budget allocations.

The relationship of the exercises to the maintenance management cycle is shown in Figure 5.

Each group is assigned a district network for which it analyzes the conditions and sets forth a program both for restoration and maintenance thereafter. Exercise 1, which emphasizes the importance of inventory, condition assessment, and data base initialization, includes the following:

- Determination of rise and fall of each link;
- Road roughness values;
- Rainfall, temperature, and duration of the rainy seasons;
- Roadway cross sections, CBR values, and structural numbers;
- Maintenance criteria and performance standards;
- Traffic distribution by vehicle type;
- Unit costs of labor, equipment and material, as well as tasks; and
- Overhead costs.

Exercises 2 and 3 emphasize the concept that the objective of maintenance is to minimize the sum of maintenance agency and user costs. This requires an optimization procedure, even

TABLE 1 TYPICAL ROADWAY CONDITION SURVEY

PAVED ROAD - ROUTE 1

KM	LEFT SIDE					PAVEMENT				RIGHT SIDE				
	DITCH		SHOULDER			Pot-	Crack-	Rut	Rough-		SHOULDER		DITCH	
	Type	Cond.	Type	Cond.	EDGE	Holes	ing	Depth	ness	EDGE	Type	Cond.	Type	Cond.
0 - 5	Gr	P	E	F	F	N	S	N	F	F	E	F	Gr	P
5 - 10	Gr	P	B	F	F	N	S	N	F	F	E	P	Gr	P
10 - 15	-	-	B	F	F	S	S	S	F	F	E	P	E	P
15 - 20	-	-	B	P	P	S	S	S	F	P	B	P	-	-
20 - 25	Gr	P	B/E	P	F	S	M/X	N	F	F	B	P	E	F
25 - 30	Gr	P	B	P	F	S	P	N	F	F	B	P	Gr	P
30 - 35	Gr	P	B	P	P	N	N	N	F	F	B	P	-	-
35 - 40	-	-	B/E	F/P	F	S	M/X	S/M	F	F	B/E	F/P	-	-
40 - 45	-	-	B	F	F	M	S	N	F	F/P	B	F	-	-

Source: Maintenance Management Case Study (ref. 2)

Legend:

Type: Gr = grass; E = earth; GV = gravel; B = bituminous; Dash (-) = non-existent

Condition: X = extensive; M = much / many; N = none; S = some

Roughness: G = good; F = fair; P = poor

if it is a heuristic one, as in the case where Exercises 2 and 3 are iterated. Exercise 2, determination of maintenance requirements, consists of two parts: restoration works (based on initial physical condition) and normal maintenance requirements (based on the interaction of vehicle usage and roadway characteristics). Exercise 2 includes estimates of the following:

- Restoration works, e.g., pothole filling and ditch lining;
- Routine maintenance requirements for Year 1 and subsequent years up to a 5-year projection;
- Increase in roughness for different maintenance strategies, e.g., 100 percent patching;
- Roughness progression of unpaved roads;
- Extent of periodic maintenance required, i.e., surface dressing, overlay, or regraveling;
- Vehicle operating costs for different levels of maintenance.

In Exercise 3, estimates of resource and budget requirements for the routine and periodic maintenance options identified in Exercise 2 are made. Between Exercises 2 and 3, delegates arrive at an optimum maintenance plan with the lowest total cost of maintenance and vehicle operation. The optimum maintenance plan is the basis for the budgetary request.

Exercise 4, balancing maintenance requirements with budget allocations, introduces the reality of limited budgets, and the effect on maintenance requirements that would yield the lowest user costs. Exercise 4 includes the following:

- Determination of priorities for recurrent and periodic maintenance in which all restorative and routine maintenance is assumed to be carried out;
- Allocation of budgeted funds to minimize the escalation in vehicle operating costs above the optimum; and
- Consideration of a do-something approach on all the maintenance requirements determined in Exercise 2

By assuming that the planned maintenance is carried out, completion of Exercise 4 sets the conditions for the beginning of Exercise 1 in the following years, using appropriate road deterioration relationships as surrogates for inspection and monitoring (Figure 5).

The coordinators work closely with the groups during the conduct of the case, providing individual and customized attention to the delegates, and tutoring when necessary. Computer-aided methodologies are available and their use is encouraged, while the conference lectures, notes, and text (*1*) provide conceptual and contextual reference. Through this close liaison between conference coordinators and delegates, the real technology training and transfer takes place.

Microcomputer Applications

For many delegates, the executive conference offers the first opportunity for hands-on experience with the computer. Computers and their utility as an effective tool in decision making are therefore introduced on the second day of the conference,

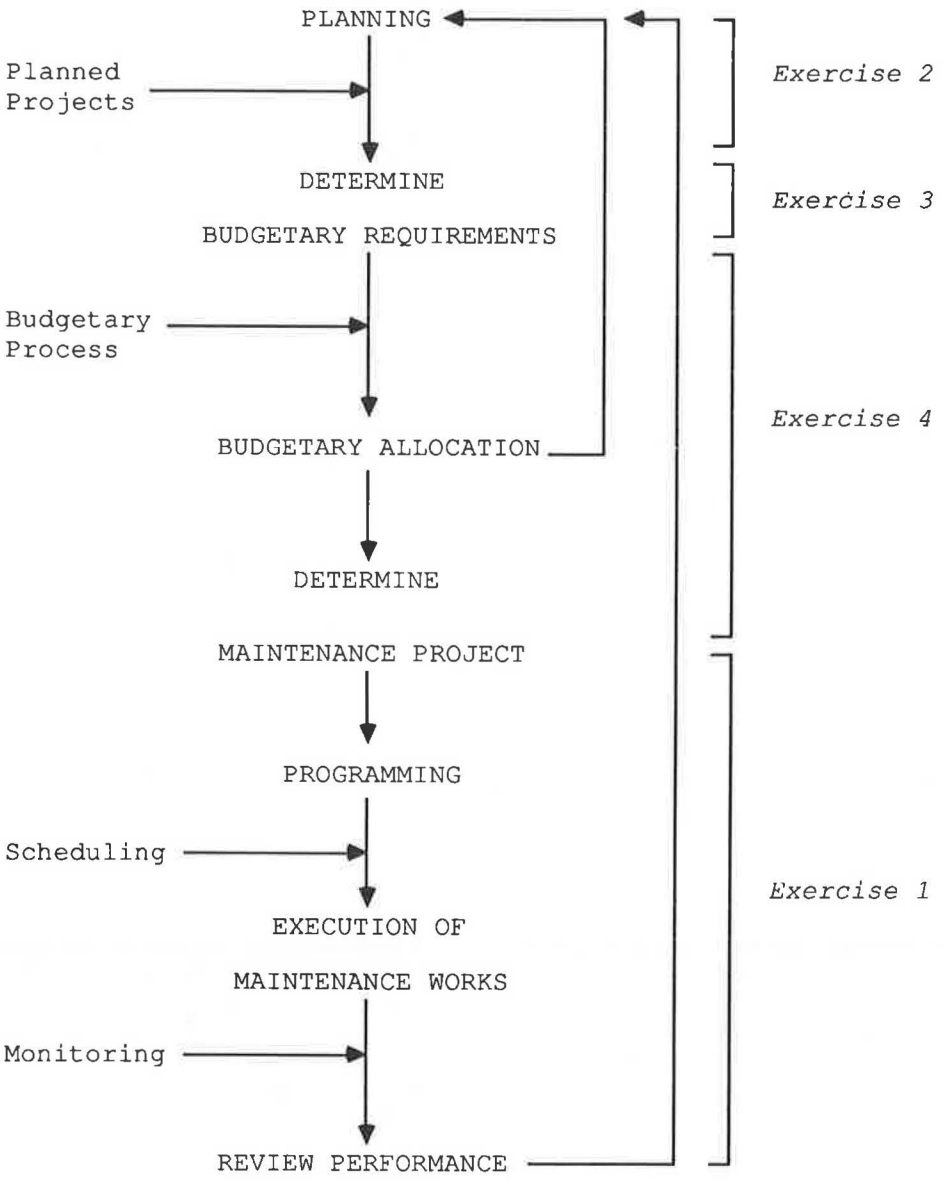


FIGURE 5 Relationship of case study exercises to maintenance management cycle.

and thereafter integrated into the presentation of all subsequent lectures and tutorial exercises. For the case study exercises that parallel the lectures, one laptop microcomputer is made available to each group, with RTIM2 and HDM–III in formats modified for enhanced user friendliness. For example, data input for RTIM2 is organized in frames as a supplement to the tutorial format in which the program is organized; the vehicle operating cost (VOC) submodel and the road deterioration submodel of HDM–III are separately provided and can be used independently, and in many ways more effectively, to solidify the concept of heuristic optimization in Exercises 2 and 3. For both models, the entire network is initialized with a default set of assumptions, some or all of which the delegates may alter to fit their own conditions. Each country delegation receives the HDM–III software and documentation, and many make disk copies of the coded networks and current solutions to continue the exercise on their return home.

The emphasis is on decision making rather than computer dexterity. As such, the case study manual (*2*) contains extensive appendices with mathematized and graphical relationships for road deterioration and VOCs. The exercises can therefore be calculator assisted, and in fact the conduct of the case insists that delegates make their initial calculations this way to develop an intuition about the likely results.

Presentation of Budgets

The case culminates in the presentation and defense of a budget by a representative from each group before a budget committee consisting of the conference coordinators, on the

final day of the conference. Following each presentation, members of the group are invited to append their comments; following all the presentations, the groups are then intensively and extensively questioned by the committee and forced to defend their budgets. It does not take long for this conference session to become serious and at times tense, as the roles are acted out, particularly when differences are pointed out between groups, each of whose districts contain the same types and lengths of road. In the end, each group is allowed to question and comment on the others' presentations, and finally the conference coordinators make their own presentation as just one model, emphasizing organization of information and analysis of alternatives. All presentations are supported by visual aids in the form of overhead transparencies.

Clearly, the case study is very much an exercise in making a case. Delegates come to appreciate the necessity to develop and justify maintenance budgets not just in terms of maintenance per se, but also in terms of the benefits of maintenance and the disbenefits of inadequate or deferred maintenance. The optimization process simulated in the exercise heightens the awareness of the idea that a given maintenance budget yields different levels of user benefits, depending on how and when the funds are expended, and establishes for the delegates the existence of an efficiency frontier (5) for maintenance.

DELEGATES' REACTIONS

In breaking the delegates into groups, two main criteria were used: geographical location where similar environmental conditions about the road network in their home countries exist; and group size that would provide maximum interaction to produce a budget to be presented to the budget committee. For example, at the third conference in 1989, 12 countries were represented by 25 delegates, who were divided into six groups. The first group consisted of the four delegates from Saudi Arabia; the second group included three delegates from Kuwait and one from the Yemen Arab Republic; the third group consisted of the three delegates from Pakistan; the fourth group included two delegates from Indonesia and two delegates from Western Samoa; the fifth group included three delegates from Tanzania and one from Kenya; and the sixth group included three delegates from the Philippines, one from Bolivia, one from El Salvador, and one from Venezuela.

In the first exercise, in which the group members attempt to define the sets of environments that describe traffic flow, climate, and roadway characteristics, the delegates are often inquisitive about how the quantity of rainfall and the duration of the rainy season affect the analysis of gravel road maintenance cycles. They begin to realize, through discussions among themselves and with the coordinators, that the assumptions made are essential for the execution of the case study, and that for the same region, road and traffic characteristics may vary substantially.

In the second and third exercises, the delegates are faced with some tough challenges related to developing a first-year maintenance plan, program, and budget. They relate well to the agency cost component of the budget, but feel that the road user cost component may not be of concern. Their rationale tends to be that budget approvals are political in nature, with decision makers being more influenced by agency immediate out-of-pocket cost. It was logical, at this point, to introduce the concept of cost trade-off between the agency costs (construction and maintenance costs) and the road user costs. The cost maintenance strategy diagram that shows the maintenance cost curve, the road user cost curve, and the total cost curve is shown in Figure 6.

During the course of 5-year budget estimation, the delegates are confronted with the complication and time consumption of road user cost calculations. Furthermore, to simulate a 5-year road deterioration process, one would need several weeks of manual calculations to successfully accomplish this task. The delegates are then provided with microcomputer systems applications for road deterioration and user cost. To their surprise, they discover that they can execute several simulations in a matter of minutes.

Each group presents its budget request, putting forward the assumptions made and its requirements for the first year and the next 5 years. After the presentations, the chairman of the budget committee informs the delegates that because of the budget constraints, their requests cannot be fully met, and they are asked to revise their recommendations to fit a specific budget figure. The group members then go back to the drawing board to come up with their adjusted 5-year programs. This process of deliberation and program adjustment brings the case study to its culmination whereby the delegates hone their understanding of the dimensions of programming and budgeting of road maintenance.

In addition to observations made by the case study coordinators, the delegates provide some remarks on the evaluation forms of the conference. Comments range from the importance of the conference in which they have participated to the usefulness of the information gained and readily applicable in practice. One delegate suggested that implementation of a road management system involving social costs and user costs augured well for the future in his country, while another delegate felt confident that he would now be able to argue effectively for an adequate maintenance budget. Certainly, the question of budget preoccupies managers and it is gratifying to note that delegates have been overwhelming in their

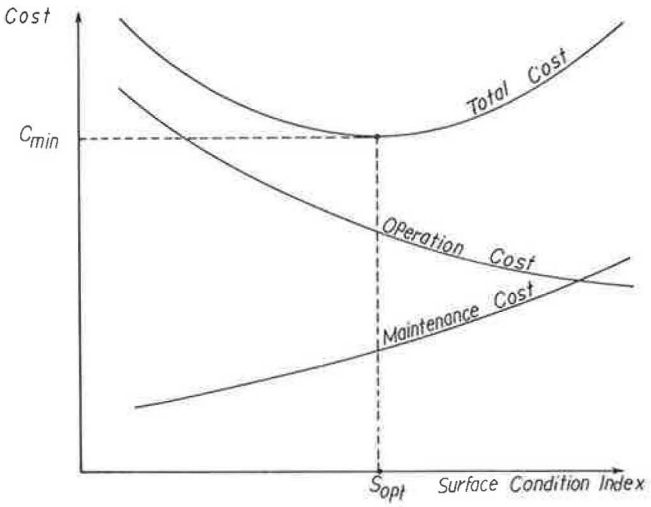


FIGURE 6 Maintenance cost strategy diagram.

expressions of the sentiment that the focus of the case study on budget preparation and defense is both well placed and effectively conducted. This tone is well captured in the following two remarks, among many, by delegates to the Third Executive Conference, which were published in the IRF World Highways newsletter and African Review (*6,7*):

> My job is preparing the annual budgets for the Ministry. Previously, I didn't think maintenance was very important, but now I think it is perhaps more important for us than construction or design. I can now discuss these procedures intelligently and explain to administrators that maintenance budgets can't be cut.
>
> This is one of the most profoundly important conferences that I have been involved in. In almost all countries, the main challenge is getting a better budget. One of my biggest problems is that I don't know what the actual highway user costs and agency operating costs are. When I have submitted proposed budgets, and have been asked what the ramifications are if I don't get the money, I haven't been able to give informed answers. Now, using the models we have developed here, I can tell the budget administrators and legislators just what it will cost the nation if we don't spend the money on maintenance.

CONCLUSION

The exit surveys of delegates to the IRF executive conferences on road management verify that the case study approach is an effective way to train personnel in maintenance management technology. In particular, by conducting the case in groups, it improves teamwork; by emphasizing submission of a budget as a major output, it improves presentation skills and capabilities of the delegates; and by a close and continuous liaison between conference coordinators and delegates, technology transfer is virtually ensured.

An on-the-job survey must still be done to finally and definitively verify the effectiveness of the case study approach. In the meantime, the conferences continue to be well subscribed, with the majority of delegates belonging to multiple delegations, representing countries that have subscribed to the program every year. A telling feature of the conference is that every year since the first one, there have been repeat delegates, a testimony indeed to the continuously evolving dynamism of the program.

Finally, the fact that to date no group has ever accepted the default conditions presented, though this would be the easiest course, is perhaps the most gratifying aspect of the exercise. Not only has it led to unique cases that cause interesting dialog, but it demonstrates how the case study approach can be instrumental in helping delegates to focus on their real problems, and also eases the process of immediate implementation on their return home.

ACKNOWLEDGMENT

The authors wish to thank the IRF for its permission to quote from its promotional material, and to make use of its pictures and slides in the presentation.

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Implementation of a Pavement Management System on Forest Service Low-Volume Roads

PALMER UTTERBACK, VIRGINIA GRILLEY, AND R. G. HICKS

The Pacific Northwest region of the USDA Forest Service manages some 3,600 mi of low-volume paved roads. The region has been faced with a need to identify procedures by which the selection and timing of maintenance activities can be ordered by priority for funding purposes. After review of available pavement management systems (PMSs), the Metropolitan Transportation Commission's PMS (MTC–PMS) was found to be appropriate for low-volume road agencies. Developed for the San Francisco Bay area, the MTC–PMS is a computer-assisted method of analyzing information about pavement condition for developing cost-effective, long-term maintenance strategies. The implementation of the MTC–PMS on low-volume paved roads within the Forest Service is described. A pilot implementation of the MTC–PMS was conducted to determine if the system could meet the pavement management needs of the Forest Service. In the summer of 1988, pavement condition data were collected on approximately 600 mi of road. A 280-mi paved road network on the Siuslaw National Forest was analyzed in detail by all of the MTC–PMS procedures. The results from the pilot implementation period indicate that the MTC–PMS can be used as an effective pavement management tool for the Forest Service.

The Pacific Northwest region of the USDA Forest Service manages an estimated network of 91,500 mi of surfaced and unsurfaced roads. Four percent of the mileage is paved, 37 percent is surfaced with aggregate, and 59 percent is unsurfaced. In 1989, an estimated $39,000,000 of appropriated and commercial use fee monies were used to maintain this investment.

Selecting road improvement programs in the Forest Service that will have the greatest benefit for the least total cost is of paramount importance, not only to meet budget constraints but also to provide the best investment for the public dollar. Many road management agencies, including the Forest Service, are considering a pavement management system (PMS) approach to help focus this complex decision process.

The principal objectives of a PMS are to identify road sections in need of treatment, predict future improvement needs, and select cost-effective alternatives. They can be used to provide a logical timing sequence for treatment needs, and to predict long-term performance consequences of management and budget decisions (*1*).

There are two levels of pavement management, the network level and the project level. Pavement management at the network level deals with the planning, programming, and budgeting decisions made for the entire highway network (*2,3*). Project level pavement management, on the other hand, deals with the specific technical requirements of individual projects.

A Construction Technology Improvement Program study, sponsored by FHWA, was initiated to determine an appropriate PMS for use on low-volume federal lands roads. Benefiting agencies represented were the Bureau of Land Management, Bureau of Indian Affairs, FHWA, National Park Service, and Forest Service.

After a review of available pavement management systems, it was recommended that the Metropolitan Transportation Commission's Pavement Management System (MTC–PMS) be adopted. This system was selected because it is simple, flexible, and is supported by a large group of agencies (*4*). The implementation of a pavement management system on low-volume paved roads in the Forest Service's Pacific Northwest region is described.

MTC–PMS OVERVIEW

The Metropolitan Transportation Commission (MTC), Oakland, California, is the transportation planning agency responsible for more than 100 cities and counties in the San Francisco Bay area. The MTC–PMS was developed in 1986 to help manage local area road networks (*5,6*). The MTC–PMS provides a computerized method for analyzing information about pavement condition and for developing cost-effective maintenance strategies (*7,8*). Adopted by over 40 California public agencies, the MTC–PMS is supported by a clear users manual, video presentations, personal telephone help, and software updates.

The methodology used by the MTC–PMS is summarized in the following sections. A more detailed description of the analysis techniques used by the MTC–PMS is found in the literature (*7–9*).

MTC–PMS Framework

The MTC–PMS is a network-level system designed for the budget planning and project identification phase of the pavement management process. The MTC–PMS is used to select and to order by priority candidate maintenance and rehabilitation projects for subsequent detailed project level evaluation. The basic process involved includes the following (*7*):

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1. Identification of the pavements to be managed,
2. Determination of current pavement condition,
3. Identification of present and future budget needs,
4. Selection of pavement sections for maintenance and rehabilitation consideration,
5. Determination of the influence of different budget levels on the network condition, and
6. Selection of the most effective maintenance and rehabilitation projects when all cannot be funded.

Inventory Data

The MTC–PMS network inventory procedure is designed to provide the road manager with information about the road network. The first phase in the inventory process is to divide the road network into uniform pavement segments. The uniform pavement segments, referred to as management sections, are those sections of paved roads that would typically perform in a similar fashion. Information collected about a particular management section includes surface type, construction date, maintenance and rehabilitation history, and functional classification. The MTC–PMS uses functional classification as a surrogate for traffic loads and structural strength (*7,8*). The functional groupings are arterial, collector, and residential. As the majority of commercial traffic on forest roads is because of timber haul, the Forest Service defined functional classification as follows:

1. Arterial—more than 50 million board feet (mmbf) hauled per year,
2. Collector—between 20 and 50 mmbf (4,000 to 10,000 trucks) per year, and
3. Residential—less than 20 mmbf per year.

The second phase in the inventory procedure is to collect information about the pavement condition of each management section. The pavement condition survey is performed by measuring the extent and severity of the following distress types (*9*):

1. Alligator cracking,
2. Block cracking,
3. Distortions,
4. Longitudinal and transverse cracking,
5. Patching and utility patching,
6. Rutting and depressions, and
7. Weathering and raveling.

Because of the excessive time and cost required to inspect each entire management section, a detailed condition survey is performed on approximately 10 percent of the area within each management section. Typically, three 200-ft long segments per mile of management section are surveyed. These short segments are known as inspection units. The condition information is used to calculate a condition score for a management section. This condition score, known as the pavement condition index (PCI), is similar to that developed by the U.S. Army Construction Engineering Research Laboratory (*10*). The PCI rating ranges from 0 to 100 as shown in Figure 1. A PCI rating of 100 defines a pavement in a new condition before the development of the first crack. The lower range of the scale, 0 to 10 (failed), represents a pavement that has failed to a disintegrated condition (*7*).

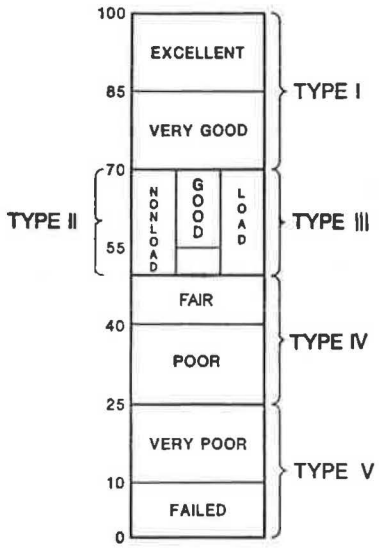

FIGURE 1 Pavement condition scale with descriptive ratings and condition categories (*9*).

Performance Prediction

To identify future budget needs, the MTC–PMS projects the PCI of each management section 5 years into the future. This feature is also used to develop alternative budget scenarios and identify long-term maintenance and rehabilitation needs.

To project the pavement condition, MTC has developed a series of family performance curves. These curves are derived from the performance of existing Bay Area pavements in terms of functional classification, surface type, PCI rating, and pavement age in years. As shown in Figure 2, the PCI rating decreases as the age of the pavement increases. There are separate curves for each combination of functional class and surface type. When the PCI rating calculated from the condition survey data does not fall directly on the family curve, the curve is adjusted to conform to the observed performance point (*8*).

Maintenance and Rehabilitation Assignment Procedure

A five-category condition definition related to the PCI scale is shown in Figure 1. Condition Category I is defined as a pavement with a PCI rating greater than 70. Category II, with the PCI ranging from 50 to 70, has primarily non-load-related distress types. Category III also has the PCI rating ranging from 50 to 70, but the distress types are primarily load related (*7*). Table 1 presents distress types that are considered to be load related and non-load-related.

The five condition categories are used for assigning user-defined maintenance and rehabilitation treatments by means of a decision tree. A typical decision tree is shown in Figure

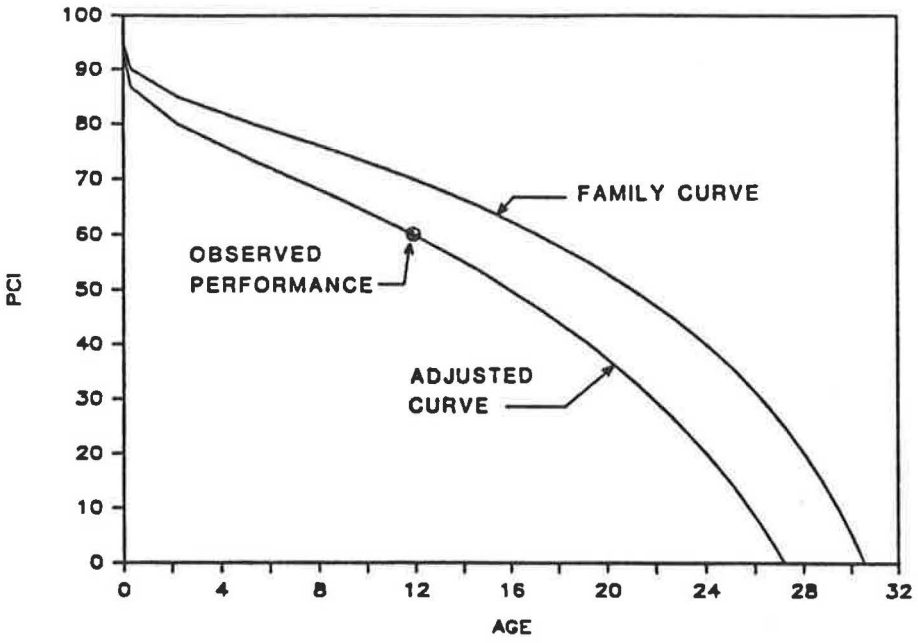

FIGURE 2 Family curve and curve adjusted for observed performance (*9*).

TABLE 1 ALLOCATION OF DISTRESS TYPE AND SEVERITY COMBINATIONS TO CAUSE OF DISTRESS (*9*)

Distress Type	Severity	Primary Cause
Alligator Cracking	All	Load
Block Cracking	Low, Medium High	Environmental Load
Distortions	All	Other
Longitudinal Cracking	Low, Medium	Environmental
Patching	All	1/2 Load & 1/2 Other
Weathering & Raveling	All	Environmental

3. Given a specific combination of functional classification, surface type, and condition category, treatments are assigned to each management section.

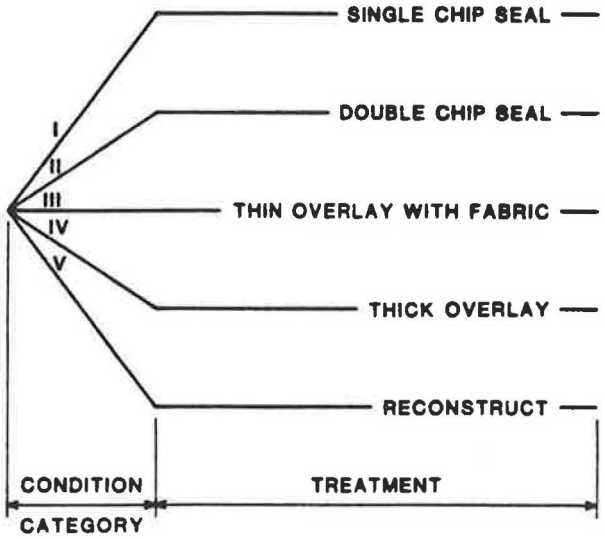

FIGURE 3 Example decision tree for a functional classification surface type combination (*9*).

Budget Analysis Techniques

Using the unit costs associated with the treatments contained in the decision tree, funding levels are determined for each year of a 5-year analysis period. The available funds are then compared with the funding requirements. MTC–PMS next orders by priority identified projects so that the most cost-effective projects are funded first (*7*).

TRIAL IMPLEMENTATION ON FOREST ROADS

Paved low-volume log haul roads have maintenance needs and conditions unique to the forest environment. It was perceived that the urban-developed MTC–PMS would require some modification to better match the maintenance and management needs of Forest Service low-volume paved roads. In the summer of 1988, the MTC–PMS was implemented on four forests in the Pacific Northwest Region to determine if the system could be adapted to suit forest conditions. The following sections describe the implementation process on one of the forests, the Siuslaw National Forest.

Siuslaw National Forest Road System

The Siuslaw National Forest is located in the coastal mountain range in the center of the state of Oregon. The Siuslaw National Forest manages approximately 2,500 mi of native, aggregate,

and asphalt-surfaced roads. Approximately 280 mi of this road system is paved with asphalt concrete. Most of the pavement was placed between the early 1970s and the mid-1980s.

Pavement Inventory

Based on the timber haul flow patterns and on information determined by interviewing the road managers on each ranger district, the Siuslaw road network was divided into 80 management sections. The average annual timber haul for the asphalt road system was estimated to be 4.6 mmbf per year, or about 1,200 trucks per year, when other road users are included. A significant portion of the timber haul occurred during the winter months when subgrade strength is the lowest. The majority of the management sections were assigned a functional classification of residential because they carried less than 20 mmbf (4,000 trucks) of haul per year. The surface types present were either asphalt concrete or an asphalt concrete overlay.

A distress survey was performed on approximately 10 percent of the surface area of each of the management sections. The distress surveys were performed in 1988 during the months of August and September and in July of 1989. A single surveyor had a production rate of 8 mi/day. With the help of an assistant, production increased to 18 mi/day.

The main cause of pavement deterioration on the Siuslaw National Forest was found to be the effects of timber haul traffic. Outside shoulder settlement, influence of environmental factors (i.e., oxidation, slope instability), chip seal raveling, surface abrasion from heavy equipment, and occasionally moss growing on pavement surfaces, were other distress types present.

The mean PCI rating of the paved roads on the Siuslaw as of September 1989 was 69 (good). Figure 4 shows the relative condition of the Siuslaw road system as a percentage of the total network surface area. Sixty percent of the road system was in very good to excellent condition with a PCI rating of 70 to 100. Thirty-five percent of the road system had a PCI rating of 50 to 69 (fair to good). Approximately 5 percent of the road system had a PCI rating of 25 to 49 (poor).

Selection of Performance Curves

There was some concern whether pavement performance curves developed from urban streets would adequately predict the performance of low-volume forest roads. Most of the Siuslaw roads were a residential functional class. Figure 5 shows an

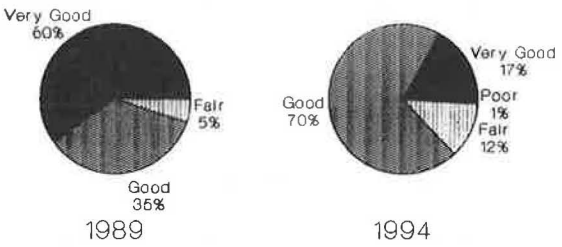

FIGURE 4 Network condition summary in 1989 and in 1994 without treatment.

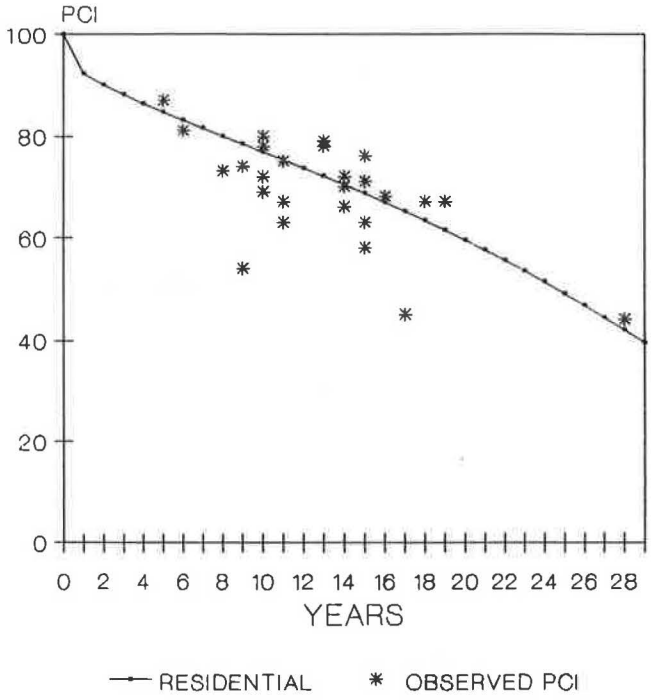

FIGURE 5 MTC family curve for residentials with asphalt surface type. The age versus PCI data points observed on the Siuslaw National Forest are plotted for visual comparison to MTC curve.

MTC–PMS performance curve along with some observed data points. Approximately 1,500 data points collected both from the Siuslaw and the Gifford Pinchot National Forests were used to determine if there was a significant difference between the observed PCI ratings and those calculated from the performance equations. A *t*-test was used to evaluate the differences between the observed data and the values predicted by the MTC–PMS. For each combination of functional classification and surface type, no significant difference was found between the means of the observed and predicted PCI ratings for a particular pavement age (V. Grilley, unpublished data).

The performance curves developed for urban traffic adequately predicted the performance on the low-volume, log haul roads of the two forests. The performance curves are also anticipated to be adequate for other forest paved-road systems with similar soils, climates, and traffic loadings.

Maintenance and Rehabilitation Assignment

The maintenance and rehabilitation treatments used on the Siuslaw differ somewhat from the treatments supplied by the MTC–PMS. Consequently, the pavement repair strategies built into the MTC–PMS default decision tree were modified to better reflect the treatments and costs currently used on the Siuslaw. The types of maintenance and rehabilitation activities commonly practiced on the forest are as follows:

1. Deep patching to correct structural deficiencies,
2. Repeated patching of shoulder settlement areas,

3. Patching with geotextile fabric,
4. Skin patching,
5. Minor pothole patching,
6. Crack sealing,
7. Chip sealing,
8. Overlay at 2 to 3 inch depth, and
9. Surface reconstruction with repair of structural base.

The timing of maintenance and rehabilitation treatments on the forest are dependent on the occurrence of timber harvest activities, spending limitations on collections in the form of road user fees, availability of capital investment funds, complaints, safety, pavement condition, and policy. Selection of maintenance and rehabilitation fixes are based on experience—i.e., what has worked in the past—and pavement evaluation techniques. Maintenance and rehabilitation activities are usually planned on a project-by-project basis.

The decision tree treatments, assignment criteria, and costs were based on an interpretation of the maintenance and rehabilitation histories of each pavement section and discussions with the engineering units on the Siuslaw National Forest. The modified decision tree developed for the Siuslaw is shown in Figure 6. The unit costs developed for the Siuslaw decision tree were based on estimates from current pavement contract

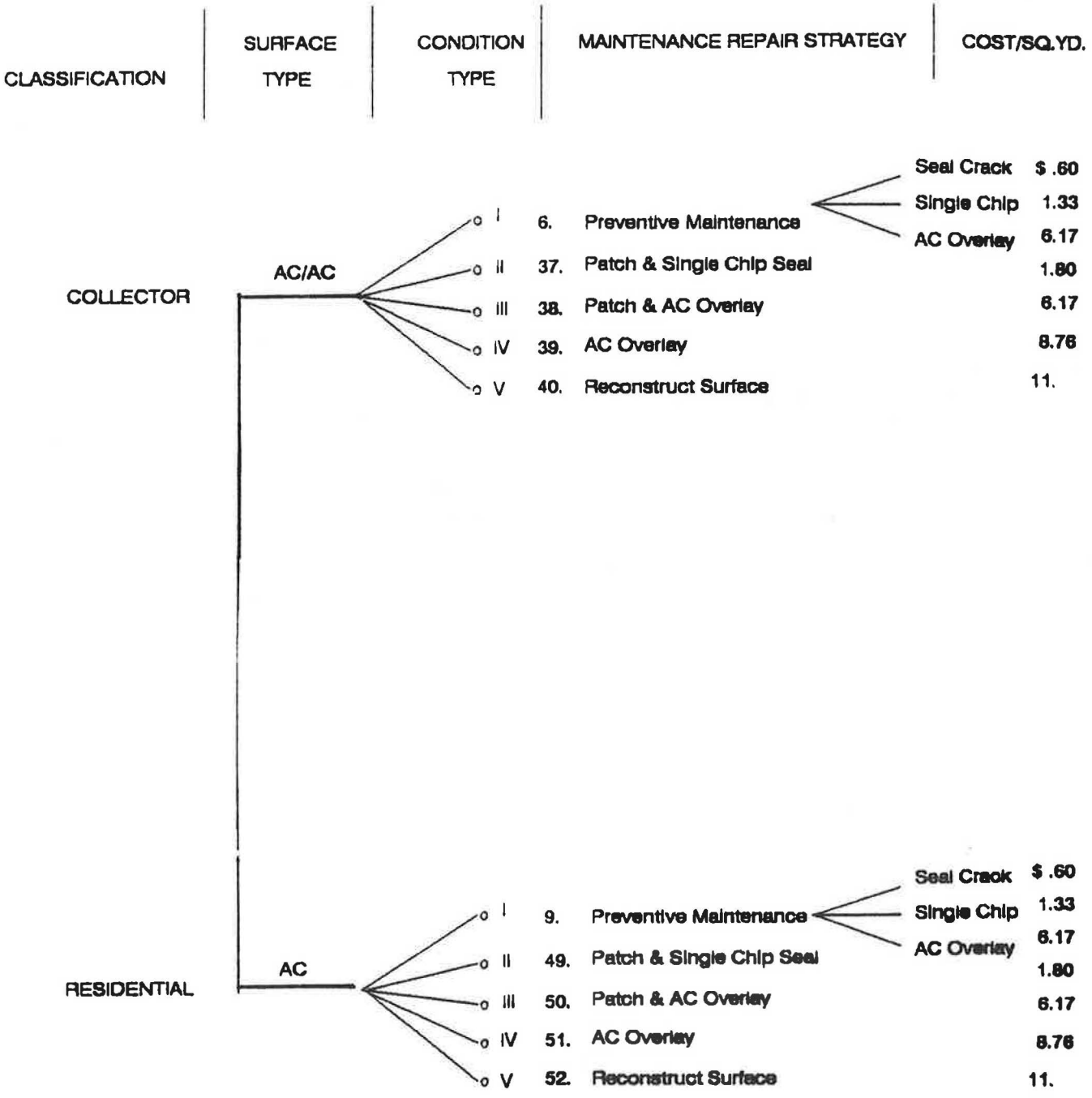

FIGURE 6 MTC–PMS decision tree modified for the Siuslaw National Forest road system.

costs occurring on the forest in 1988 and 1989. MTC–PMS default treatments and costs were used in cases when local information was not readily available.

Budgeting Alternatives

As shown in Figure 4, 60 percent of the Siuslaw network was in the very good condition category in 1989. With no maintenance or rehabilitation, it was predicted that the percentage in the very good category would fall to 17 percent by 1994. Only 5 percent of the pavements fell into the fair category in 1989; this increased to 12 percent in 1994. Without maintenance or rehabilitation, the Siuslaw network average PCI rating was predicted to fall from 69 to 58 within the 5-year analysis period.

If all the treatments assigned by the decision tree were applied, the average PCI for the Siuslaw network would increase to 80 by 1994. The cost of applying these treatments would be $7.7 million.

Long-Term Strategies

An analysis was performed to determine what the most cost-efficient strategy would be over a 25-year life. A 110-mi representative sample of the 280-mi road system was selected for a detailed alternative analysis. Table 2 is a summary of the alternatives and their costs.

The do-nothing strategy (Alternative 1) results in zero maintenance costs. However, Figure 7 shows that the Siuslaw road system deteriorates to a failed condition over a 25-year period. As more pavements approach a failed condition, unsafe driving conditions and user costs are likely to increase significantly, a situation considered unacceptable for the Siuslaw.

Alternative 2 involves managing the Siuslaw road system for a network mean PCI rating of 70. This procedure resulted in the lowest acceptable equivalent uniform annual cost (EUAC) for a 20-year period. Managing the network for this condition level typically avoids allowing pavement sections to deteriorate such that reconstruction is needed. This alternative also avoids overmaintaining the road system. The advantage of this strategy is that higher-priority roads can receive the necessary treatment and promote cost-effective expenditure of road maintenance funds. This alternative was the preferred one.

TABLE 2 ALTERNATIVE MANAGEMENT STRATEGIES

Alternative	Management Strategy	EUAC
1	Do Nothing	0
2	Maintain Mean Network PCI of 70	$250,000
3	Maintain Mean Network PCI of 80	$300,000
4	Reconstruct Pavements The Year After They Reach a PCI of 25	$400,000

Alternative 3 involved maintaining the network at an average PCI rating of 80. Maintaining the network to this extent would cost more than 20 percent more on a yearly basis than to manage to a PCI rating of 70. The benefits of increasing the PCI rating from 70 to 80 do not appear to be justified in terms of cost.

Alternative 4 resulted in the highest EUAC for the 25-year analysis period. In this alternative, each pavement is reconstructed only after failure (PCI = 25). The PCI trend for this strategy is presented in Figure 8. The mean network PCI rating appears to reach a low of 50 (fair) but rising to 70 (very good) at the end of the 25-year period. However, because of this management strategy, there are always a number of roads approaching a failed condition. This strategy may result in some roads being in a condition unacceptable for timber haul or recreation use.

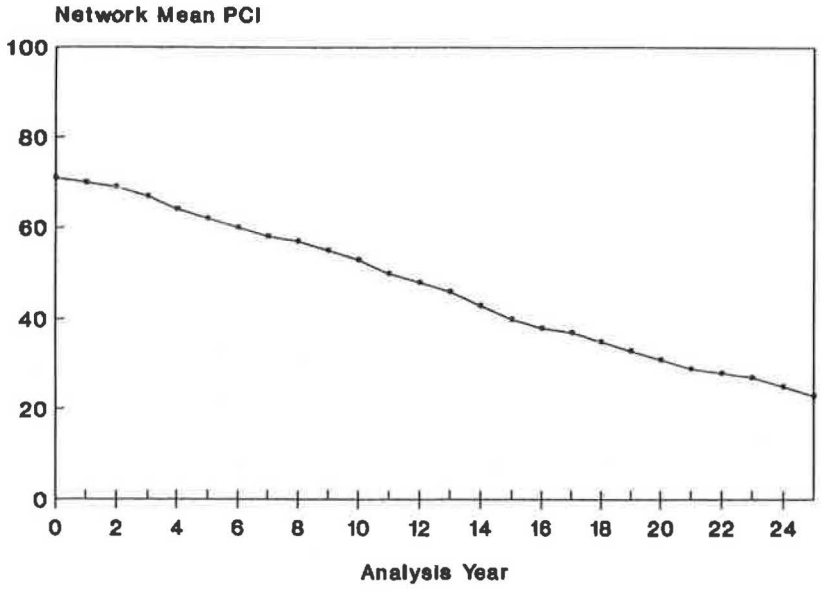

FIGURE 7 Mean PCI rating for the do-nothing strategy.

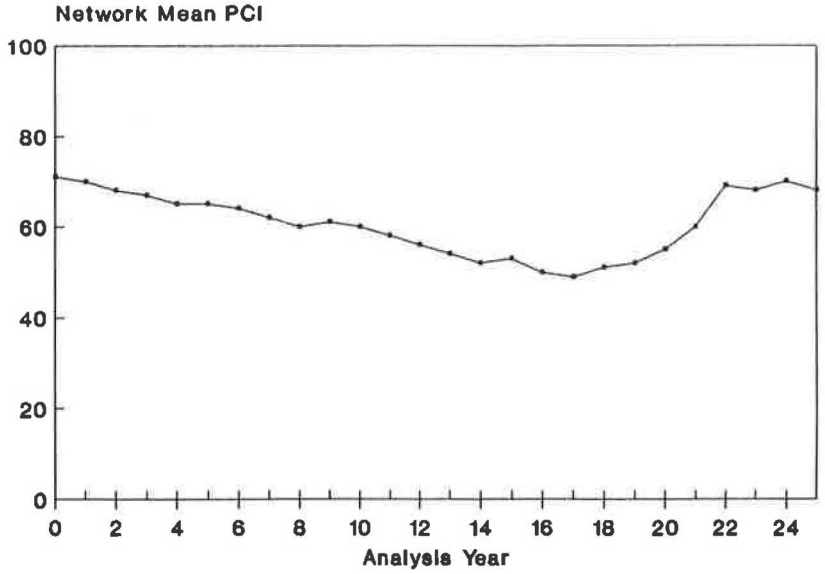

FIGURE 8 PCI rating trend when the pavements are reconstructed after failure.

CONCLUSIONS

On the basis of the experience with the Siuslaw and other forests, it can be concluded that the MTC–PMS possesses the features necessary to be a useful pavement management tool for the Forest Service. The usefulness of the MTC–PMS as a tool will depend on how well experience and good engineering judgment are blended with the computing capabilities provided by the software. Effective pavement management in the Forest Service will require the coordinated input from many sources, including

1. The engineering judgment and expertise of the field personnel who drive and care for the roads on a daily basis,
2. Budgeting personnel who use the MTC–PMS as a long-range planning and budgeting tool, and
3. Pavement engineers who can conduct project level analyses once the network level input is supplied by MTC–PMS.

Benefits Provided by MTC–PMS

Some of the features provided by the MTC–PMS that can be used to advantage by the Forest Service include the following:

1. The system provides the Forest Service with better inventory data on the paved road system.
2. The MTC–PMS provides a consistent and objective procedure for rating the condition of each pavement section. Use of the PCI rating scale can lead to better communication among Forest Service road management personnel. The PCI rating scale can also enhance communication between engineering and other Forest Service disciplines in regards to explaining the condition and needs of the asphalt paved road system. The PCI rating scale can also be used to help explain the asphalt road management policy to the public, which is increasingly using the Forest Service road system.
3. The system provides an analytical method for identifying, ordering by priority, and justifying pavement rehabilitation and maintenance proposals.
4. The MTC–PMS provides a tool that can evaluate the future impact of budgeting decisions made in the present. Impact can be measured in terms of the change in PCI value, deferred costs, and distress.
5. The system provides a means for monitoring or tracking the condition and cost of the paved road system over an extended period of time. Although it was found that the performance equations contained within the MTC–PMS adequately predicted the performance of low-volume roads with seasonal traffic, this information would be useful for project level analysis.

Limitations When Applied to a National Forest Road System

A few features of the MTC–PMS were found to be incompatible with some of the conditions found in the National Forest road system. Some of these conditions include

1. Outside shoulder settlement distress areas are not fully accounted for in the distress survey procedure. Some interpretation is required.
2. The program is somewhat sensitive to data input errors. The developers of the program are in the process of fixing this problem.

CONTINUING EFFORTS

Eleven out of the 19 national forests in the Pacific Northwest region have begun implementing the MTC–PMS. The remaining eight forests have less than 100 mi of paved road each. By March 1990, most forests have completed the inventory proc-

ess and are beginning to determine what maintenance and rehabilitation treatments are needed on their road networks. The following recommendations apply to the continuing implementation effort currently underway in the Pacific Northwest region.

1. Provide a format for cooperation and sharing of information, techniques, and expertise among PMS users.
2. Develop auxiliary data bases for each forest to track traffic, timber haul, season of haul, pavement condition (PCI rating), treatments, and costs over an extended period of time for each pavement section. Use these data to perform project level analyses and to set road user fees.

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Maintenance of Unpaved Roads in Wet Climates

T. E. JONES AND Y. PROMPRASITH

In 1987, a research project on gravel roads was established in Thailand between the Thailand Office of Accelerated Rural Development (ARD) and the Overseas Unit of the U.K. Transport and Road Research Laboratory (TRRL). The principal objective was to compare the effects of alternative maintenance strategies for unpaved roads in wet regions. The project also studied the relative performances of motor graders and tractor-towed graders for maintaining roads carrying traffic volumes of 50 to 300 vehicles per day. The potential benefits are the optimization of grading frequencies for different levels of traffic. In addition, the opportunity was taken to evaluate the applicability of TRRL prediction models for gravel road deterioration produced from data obtained in dry and temperate regions to roads in wet climates. Preliminary results from the study suggest that tractor-towed graders can achieve similar reductions in roughness to those obtained with the motor grader. Running costs of the tractor-towed grader units are less than half those of the motor graders. If investment and depreciation costs are considered, then the savings in maintenance costs are substantially higher. An additional problem in Thailand is that of maintaining rural roads built in hilly terrain with gradients up to 17 percent. The ARD has investigated methods of reducing the high maintenance costs on such roads and found that the most cost-effective solution was to surface the most vulnerable lengths of road with concrete blocks. After 2 years' trafficking, this treatment has proved to be highly cost-effective, requiring no maintenance despite heavy rainfall.

Over the past 10 to 15 years, the Overseas Unit of the U.K. Transport and Road Research Laboratory (TRRL) has carried out extensive research on unpaved road deterioration and maintenance in several tropical countries. The results of these studies have led to the development of models for predicting the rate of various forms of deterioration and a means of optimizing maintenance strategies (*1*).

However, the climate in the countries where research was undertaken has been predominantly dry or temperate. There has, therefore, been a need to validate the prediction models in more extreme climates, particularly in countries with annual rainfall above 2000 mm.

In 1987, a collaborative research project was established in Thailand between the TRRL and the Office of Accelerated Rural Development (ARD) of the Ministry of Interior, Thailand. One of the principal aims of the ARD is to provide an all-weather road network in rural areas and to date it has constructed over 19 000 km of gravel roads in some 73 provinces. The ARD is also responsible for the maintenance of these roads and looks to this research study to determine optimum grading frequencies and to evaluate appropriate equipment to achieve reductions in overall maintenance costs. Current progress on this work is reported.

The ARD is also currently undertaking a project to establish the costs and performance of precast concrete block paving. This technique is now well established in Europe and North America but its use as a surfacing for low-cost rural roads is unusual and if successful would provide an alternative maintenance strategy.

The objectives of the ARD/TRRL research study are as follows:

1. To verify prediction models of gravel road deterioration produced from data obtained in dry and temperate regions under extreme conditions of climate. A minimum requirement is that the existing TRRL models are calibrated for wet climates, but probably new prediction models will be developed.
2. To study the relative merits of different maintenance strategies for unpaved roads in wet regions. This project would enable the TRRL to complete its studies on the performance of gravel roads over a wide range of climates. It has already been established that climate significantly influences the rate of deterioration but there is a lack of quantified data available on gravel road performance under very wet conditions.

Thailand provides an excellent environment for this research project. The rural development networks have roads carrying 50 to 400 vehicles per day (vpd), and in specific geographic areas annual rainfall is in the range 1700 to 2400 mm. In addition, there is a well-established road organization responsible for maintenance.

The potential benefits of this research study to the ARD and other organizations are as follows:

1. The study should enable the maintenance organization to determine optimum grading frequencies, which would result in more cost-effective maintenance, reducing overall maintenance costs and enabling additional funds to be spent on upgrading or construction projects. In addition, by quantifying the changes in surface condition, improved maintenance strategies could be obtained.
2. ARD should be able to evaluate the merits of alternative equipment such as tractor-towed graders and drags to reduce maintenance costs on minor roads.
3. The study should enable the ARD to identify the effect of rates of change in surface condition on vehicle operating costs. This would be achieved by developing prediction models based on the field data and incorporating them in the TRRL microcomputer program RTIM2 (*2*), which evaluates the costs and benefits of different maintenance strategies. This infor-

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mation would be invaluable to support requests for maintenance funds from Central Government.

LOCATION OF SITES

Two provinces incorporating elements of the ARD road network are included in the research study. These are Hatyai in the southern region, where the study became operational in July 1987, and Nakhon Phanom in the northeast region where monitoring started in April 1988.

In both provinces, there are 32 test sections, each of which is 500 m in length. All test sections are located on existing roads. The average annual rainfall based on 30 years of records is approximately 2400 mm in each province. There are, however, specific differences in traffic volumes and loading and also in the materials available for gravel wearing courses.

At Hatyai, the traffic varies from 50 to 300 vpd with no heavy vehicles using the test roads, whereas at Nakhon Phanom the traffic varies from 25 to 250 vpd with substantial volumes of heavy traffic on one of the test routes. Materials used for gravel wearing courses in the Hatyai region comprise sandstones, volcanic tuffs, laterites, and limestones, whereas in the Nakhon Phanom region the gravels are derived predominantly from red sandstones of the Maha Sarakan formation together with terraced Mae Khong alluvial sandstones and siltstones.

MEASUREMENT OF EXPERIMENTAL VARIABLES

The principal experimental variables measured in the study are as follows:

- Surface roughness,
- Rut depth,
- Traffic volume and loading,
- Climate,
- Gravel loss,
- Operating costs,
- Material properties (measured both in the laboratory and in-situ), and
- Grading frequency.

Because of their geographic locations, the majority of the ARD road networks have large-radii horizontal curves and vertical gradients of less than 2 percent. Therefore, as the road alignment is similar in most cases, alignment was not included as a variable in this research project. The experimental framework used in the study is presented in Table 1.

The basic framework has 24 experimental sections 500 m long. In two provinces, groups of eight test sections have been established on three different roads carrying the traffic levels presented in Table 1. Each road, therefore, includes three levels of maintenance using a towed or motor grader and two nil maintenance sections. The resultant 48 test sections have been supplemented in both provinces by a further group of eight test sections incorporating a different gravel wearing course but utilizing the same range of maintenance strategies and equipment resulting in an overall total of 64 test sections.

Surface Roughness

Measurements of surface roughness were made using vehicle-mounted bump integrators. These units are calibrated every 6 months against the Abay calibration beam (*3*) over roads in good structural condition but exhibiting a wide spectrum of roughness levels. In the study, three measurements of surface roughness are taken in each traffic lane. Frequency of measurement has been established at monthly intervals and immediately before and after grading operations.

Rut Depth

The maximum rut depth is recorded in each wheeltrack in each lane utilizing a 2-m straight edge and a wedge calibrated in millimeters. Ten measurements are taken in each test sec-

TABLE 1 EXPERIMENTAL FRAMEWORK

	Traffic			
	< 75 vehicles per day	75-150 vehicles per day	150-300 vehicles per day	
	Grading frequency per year			
Vertical gradients < 2 per cent	3 2 1 0	3 2 1 0	3 2 1 0	Annual rainfall 2400mm
No of sections	8	8	8	

Duplicate sections are maintained either by motor grader or towed-grader

tion at 100-m intervals concurrently with the monthly roughness measurements.

Traffic Volumes

The traffic volumes on each road incorporating the test sections are measured with automatic inductive loop counters. The counters, using loops buried beneath the gravel wearing course, are read manually each day. They also incorporate loggers that store data for periods of up to 6 months and have the facility to back-analyze. The counter readings are augmented by manual classified traffic counts taken at six monthly intervals.

Traffic Loading

Axle load surveys are carried out at 6-month intervals utilizing the TRRL portable weighbridge. Classified traffic counts are taken concurrently with the axle load surveys.

Climate

A volumetric rain gauge and an automatic intensity rainfall recorder are located on each experimental road. The volumetric rain gauges are read manually each morning, whereas the automatic recorders incorporate data cassettes, which are renewed each month for analysis by microcomputer.

Gravel Loss

Gravel loss measurements in terms of remaining thickness of gravel wearing course are taken concurrently with the roughness and rut depth measurements. Additional measurements are taken when carrying out pavement strength evaluations using the dynamic cone penetrometer (DCP) (*4,5*).

Operating Costs

During each maintenance activity, the operating costs of the equipment are monitored, including the motor grader, tractor and towed-grader, compaction equipment, and water tanker. As the study proceeds, data on vehicle operating costs for the traffic using the rural networks will be collected. These data will then be used to evaluate the vehicle operating costs relevant to different maintenance strategies and surface conditions.

Laboratory Testing

At the beginning of the study, samples of gravel wearing course and subgrade were taken from each section and subjected to laboratory tests. The tests included the measurement of abrasion values, an important parameter in terms of gravel wearing course performance.

In Situ Testing of Materials

At six monthly intervals, before and after the rainy season in both provinces, in situ measurements of material strengths are carried out using the DCP.

RESULTS

Data from Hatyai Province now cover a 2-year period, while 12 months' data have been collected from the Nakhon Phanom Province sites. The monitoring period will cover a period of 3 years. The large quantity of data has not yet been fully analyzed, and this chapter summarizes the more important results to date from both sites.

Roughness

All test sections were surface-graded initially to enable measurements of deterioration to start from the same base. The initial roughness levels of the sections were in the range 2800 to 3200 mm/km. The surface roughness measurements on the Hatyai test roads did not reach levels higher than 6000 mm/km during the first 12 months. This result is partly because of the good condition of the roads, including side drainage, before the onset of the study but also because the rainfall was below average. During the second 12-month period at Hatyai, levels of roughness were up to 20 percent higher, while the rainfall was close to the 30-year average.

Another factor influencing the rate of change of roughness is traffic. Although there is an ADT of 290 vpd on one of the roads, the traffic loading is relatively light. This was the reason for identifying a specific road carrying medium and heavy trucks in the Nakhon Phanom region for inclusion in the study.

Generally, both the towed and motor graders reduce surface roughness by about 30 percent to a level of 3500 mm/km, as shown in Figure 1. However, the roads are still in moderately good condition and it remains to be seen whether, as the roads deteriorate, similar reductions in surface roughness can be achieved by both types of equipment.

The final prediction models for estimating changes in roughness will incorporate measurements of the physical properties of the materials including abrasion and plasticity. These two variables must influence the development of roughness, and laboratory tests are being carried out on each material type as the study proceeds.

Rut Depth

The development of rutting on four consecutive sections in Hatyai is shown in Figure 2. The towed grader was used for maintaining these sections and it can be seen that rut depth after grading was reduced to 5 to 7 mm. This is a good standard for an unpaved road; however, each ARD maintenance operation involves the use of water distributor when necessary and always compacted with a roller, which partly accounts for the low levels of rut depth achieved after maintenance had been carried out. Similar results were obtained when the motor

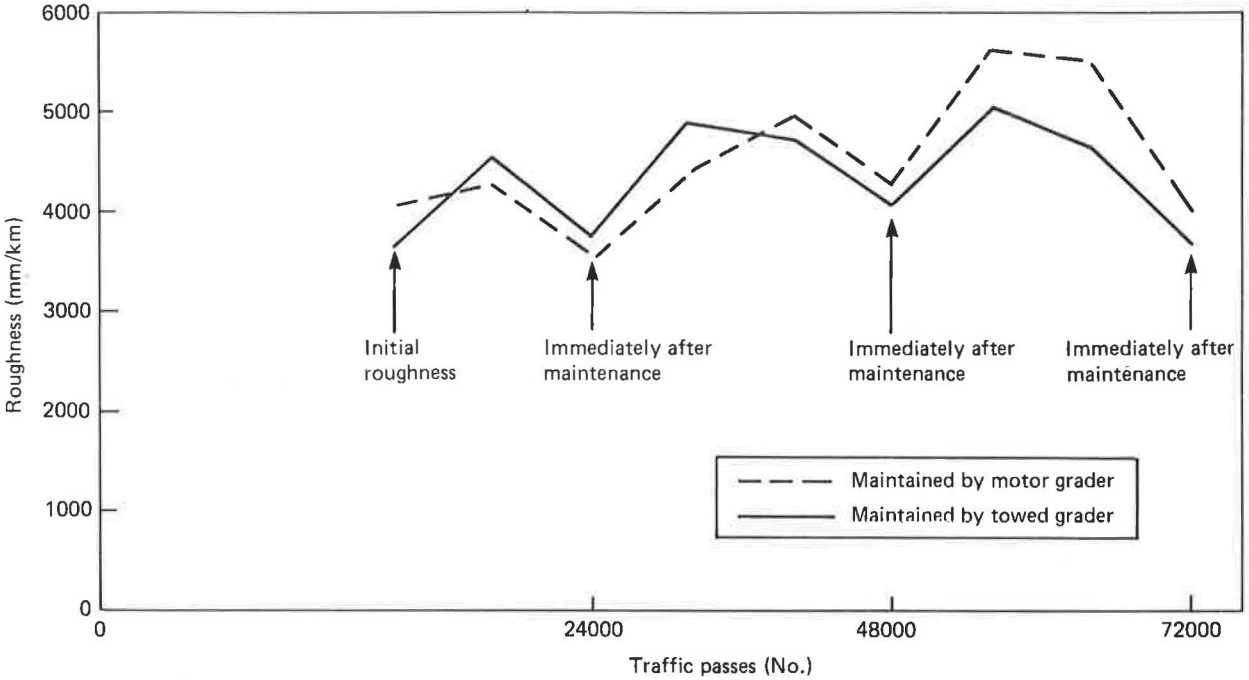

FIGURE 1 Examples of reductions in roughness achieved with motor grader on two trial sections.

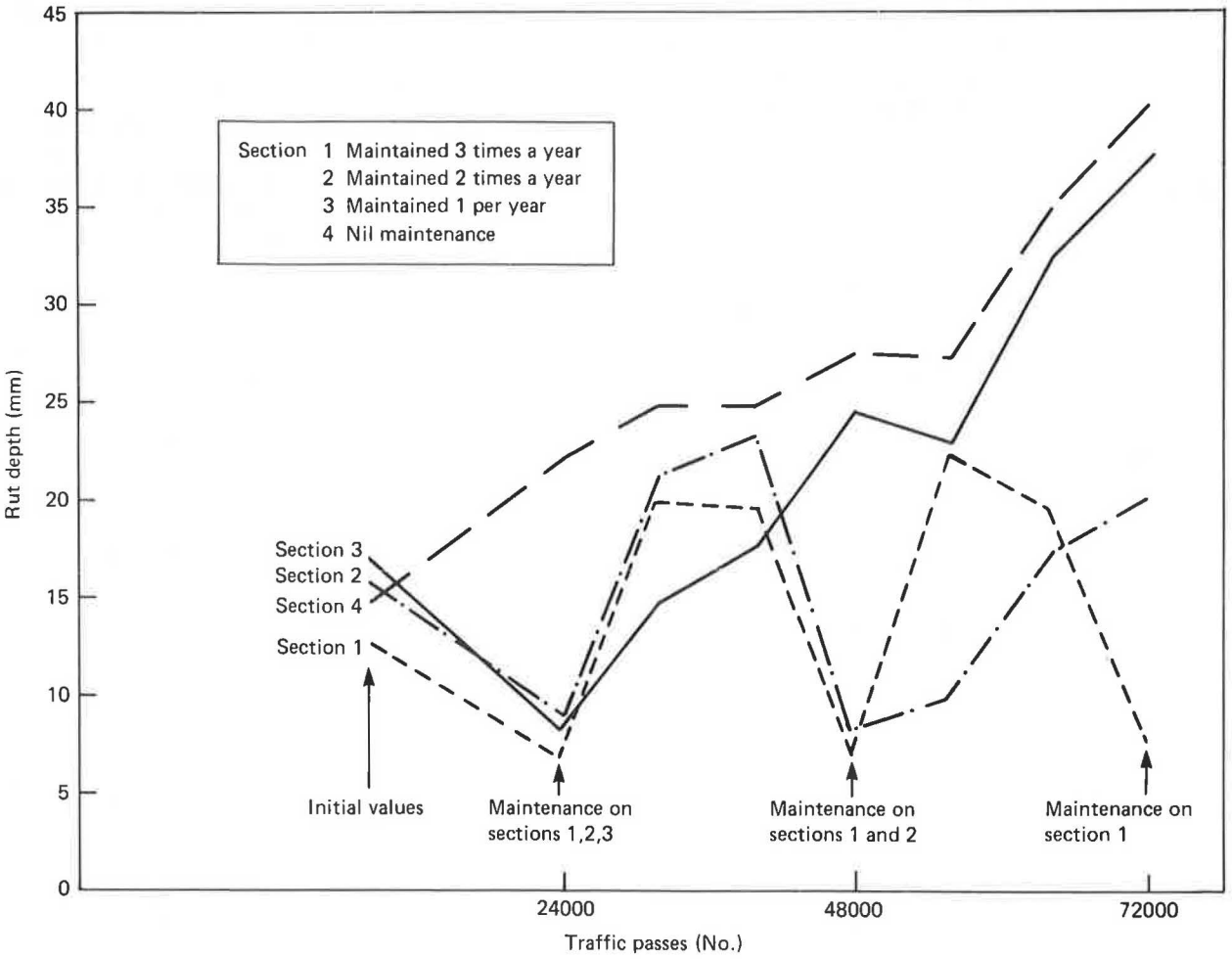

FIGURE 2 Examples of reductions in rut depth achieved with different grading frequencies (towed grader).

grader was used. The rut depths reported are the average of 10 measurements in each section remeasured every 6 weeks. On the nil maintained sections, rut depths have not been allowed to develop beyond 80 mm because of the potential danger to traffic. When this level has been reached, the section is graded and the progression of deterioration begins a new cycle.

Costs of Operating Maintenance Equipment

One of the objectives of the research is to evaluate the potential saving using the towed grader instead of the motor grader. Table 2 presents the current costs to ARD of the equipment presently used for maintenance.

The costs are much lower than found elsewhere (*1*). The low cost of investment and depreciation reflects the age of the fleet in which most of the motor graders are 15 years old and some are 22 years old. The low repair and maintenance costs reflect the high standard of expertise in the ARD workshops. It is too early to predict accurately what the running costs of the towed grader in Thailand will be. However, during each maintenance activity detailed records are kept of all expenditure and consumption, particularly of fuel, which is measured accurately with a calibrated pump. Currently, when the maintenance activity involves a towed grader, the costs are less than half that of the same activity when the motor grader is used. The current ARD allocation for maintenance is 12,000 Baht/km or U.S. $500/km.

Traffic

Table 3 presents the estimated ADT on the basis of the automatic traffic counter results.

The maximum axle load found during the first survey was 9.1 tonnes, but the average axle loads for trucks using the ARD road are usually between 5 and 6 tonnes. The maximum traffic loading is expected to be not greater than 5,000 standard axles per year for each road. Further surveys are carried out in the wet season to detect any changes in vehicle type, commodities, and axle loads.

Climate

The first 12 months' rainfall was below the predicted average of 2300 mm. From May 1987 to April 1988 only 1550 mm was recorded although over 500 mm of rain fell during December 1987. There were some technical problems with the automatic rainfall recorders partly because of high temperature and humidity but they have been resolved. The 1988/1989 rainfall period experienced levels of rainfall nearer the predicted average. Rainfall in 1989/1990 was higher than normal.

Gravel Loss

Gravel loss has been measured in each section. The average losses for 1 year are presented in Table 4.

TABLE 2 OPERATING COSTS OF EQUIPMENT

Plant type	Costs per hour US$							
	Investment	Dep.	Fuel	Maint.	Tyres	Repair	Operator	Total
140/160 hp motor grader	1.51	2.67	5.12	1.08	1.10	4.28	1.20	16.97
120/140 hp motor grader	1.07	1.88	4.27	0.90	1.10	3.05	1.20	13.48
10 ton roller	0.56	0.98	2.73	0.45	1.00	1.90	0.60	8.24
65/80 hp tractor	0.20	0.35	2.47	0.44	0.59	0.55	0.60	5.19
water tanker	0.31	0.54	4.52	0.48	0.60	1.07	0.60	8.12

TABLE 3 ESTIMATED ADT ON EXPERIMENTAL SECTIONS IN HATYAI

Road No.	Mean ADT (2 way)	Standard Deviation
11010 (site 1)	287	82
11004	150	46
11010 (site 2)	90	27
11011	40	16
11012	38	16

TABLE 4 ANNUAL GRAVEL LOSS

Road No.	Mean ADT (2 way)	Gravel Loss (mm)
11010	287	21
11012	38	8
11004	150	15
11011	40	12

Testing of Materials

All sections were sampled at the start of the project and the tests have been repeated during the study. One of the objectives of these measurements is to identify any changes in the physical properties of the materials as a result of trafficking and climate. The initial laboratory results are presented in Table 5.

In Situ Strength Tests

The effective strength of an unpaved road varies daily because of changes in the moisture content, which is a function of permeability and rainfall. The U.S. Waterway Experimental Station has carried out considerable research on the trafficability of different soils (*6*). The results showed that if a road has a California bearing ratio (CBR) greater than 12 percent it is capable of supporting 50,000 vehicle passes with tire pressures not greater than 50 N/cm^2 and axle loads of 8.2 tonnes. The failure criterion for this analysis was a rut depth of 75 mm. Therefore, this level of bearing capacity should be adequate for most of the unpaved ARD rural roads in Thailand where tire pressures and axle loads are likely to be less than the U.S. values. The ARD-specified gravel thickness for new construction is 200 mm for their standard road. Table 6 presents the in situ CBR values on the basis of dynamic cone penetrometer (DCP) tests on the Hatyai sections, which indicate that, on this basis, strengths are adequate. Further measurements were taken during the wet season to detect any changes that may occur, but preliminary analysis indicates that, apart from the top 50 mm, changes of strength within the road are minimal.

INTERLOCKING CONCRETE BLOCK PAVEMENT (ICB)

The maintenance of unpaved roads can, in some circumstances, become uneconomic, even when grading frequency and other maintenance tasks have been correctly optimized. This may be the case, for example, where steep gradients are unavoidable or where particularly weak subgrades require large quantities of imported material to retain an acceptable running surface.

An unpaved road may also be unacceptable when it passes through a large village because of the accelerated deterioration caused by increased traffic movements and the nuisance created by dust in dry weather.

The usual remedy for these problems in the past has been the application of a double surface dressing. However, since 1986 ARD has experimented with interlocking concrete block (ICB) paving as a potentially more cost-effective solution.

TABLE 5 RESULTS OF INITIAL LABORATORY RESULTS ON SURFACING MATERIALS

Sieve size	Grading: Retained Material (%)							LL	PI
	1"	3/4"	3/8"	No 4	No 10	No 40	No 200		
Road 11010	-	97	77	51	35	29	14	34	10
Road 11004	-	-	99	92	69	47	23	33	11
Road 11012	-	93	74	59	48	43	21	30	10
Road 11011	-	88	67	42	31	29	21	32	9

The normal criterion used by ARD for PI is that it should be in the range 6-12.

TABLE 6 CALCULATED CBR VALUES BASED ON DCP RESULTS

Road No	Average CBR (%)	Penetration depth (mm)
11010 (site 1)	46	0-192
	29	192-500
11004	37	0-242
	12	242-500
11010 (site 2)	41	0-200
	44	200-500
11011	17	0-300
	11	300-500
11012	19	0-165
	28	165-500

Summary of Benefits

The benefits of ICB pavements compared with bituminous surface treatment are as follows:

1. ICB pavements require minimized capital investment in heavy equipment.
2. ICB pavements can eliminate the importation of expensive raw materials such as asphalt and fuel.
3. Thailand has a surplus of locally produced cement, and ICB pavements provide alternative uses for this material.
4. The discounted costs of an ICB surfacing over 20 years is about one-third less than a double bituminous surface treatment.
5. ICB pavements can be constructed by unskilled laborers under moderate supervision, whereas bituminous pavements need to be constructed by skilled technicians with high levels of supervision.
6. An ICB of 80-mm web thickness can support wheel loads up to 5000 kg under the worst conditions, such as saturated and weak subgrades and poorly drained materials. Tables 7–9 present results from research done by the Engineering and Technology Faculty in Bangkok (*7*) under varying conditions of soil type and traffic.
7. Lower labor costs through the use of relatively unskilled labor.
8. The pavement surface can be removed and reused elsewhere if required.
9. Maintenance cost of ICB pavement is lower than bituminous surface treatment.
10. ICB pavement provides employment opportunities for local villagers who quickly acquire the necessary skills. For ICB surfacing, labor costs are typically 25 percent of the total project cost. For a double bituminous surface treatment, labor costs are typically 2 percent of total project cost, and this labor is imported.

Experience suggests, however, that ICB pavement is unsuitable where traffic speed regularly exceeds 60 km/hr, and also that construction is slower than for a double bituminous surface treatment.

Economic Advantages of ICB Pavement

The economic benefits from substituting an ICB pavement for a double bituminous surface treatment were based on the following assumptions:

1. A reduction in the initial construction cost of U.S. $900/km.
2. A reduction in the average annual maintenance cost of U.S. $470/km.
3. Resealing the bituminous surface every 7 years.

The first ICB pavement constructed in Thailand was near Chiang Mai. It was constructed in 1986 by the villagers under

TABLE 7 PERFORMANCE OF ICB PAVEMENT, 50 mm THICK, 1988

No of vehicle passes	Maximum rut depth (mm)			Pavement area cracked (%)	Remarks
	Left Track	Average	Right Track		
25	24.20	17.15	10.10	16.09	
50	38.40	24.85	11.30	34.62	Average temp 32°C
100	46.30	29.20	12.10	47.79	Tyre pressure 80 psi
300	69.40	45.90	22.40	97.32	Average speed 7.2km/hr
400	90.10	59.25	28.40	100	Gross vehicle weight
500	82.50	61.75	41.00	100	= 13,600 kgs
800	131.50	95.85	60.20	100	(4 wheel truck)
1000	142.20	97.35	52.50	100	

TABLE 8 PERFORMANCE OF ICB PAVEMENT, 80 mm THICK, 1988

No of vehicle passes	Maximum rut depth (mm)			Pavement area cracked (%)	Remarks
	Left Track	Average	Right Track		
100	11.80	6.80	1.80	1.83	
200	8.51	5.86	3.20	2.85	
300	9.30	6.50	3.70	4.09	
400	14.60	10.06	5.51	6.62	
500	18.10	12.30	6.50	8.82	
600	23.51	15.56	7.61	9.43	Average temp 32°C
700	28.31	18.56	8.80	9.65	Tyre pressure 80 psi
800	29.10	19.10	9.10	10.23	Average speed 7.2km/hr
900	33.50	21.51	9.51	10.89	Gross vehicle weight
1000	34.00	22.85	11.70	11.40	= 13,600 kgs
1100	34.20	22.71	11.21	11.98	(4 wheel truck)
1200	55.00	33.26	11.51	12.50	
1300	60.01	35.91	11.80	13.15	
1400	62.00	37.00	12.00	13.60	
1500	55.40	34.25	13.10	14.33	
1600	51.51	32.36	13.21	14.84	
1800	76.70	45.05	13.40	15.78	

TABLE 9 PERFORMANCE OF ICB PAVEMENT, 100 mm THICK, 1988

No of vehicle passes	Maximum rut depth (mm)			Pavement area cracked (%)	Remarks
	Left Track	Average	Right Track		
100	3.00	2.20	1.40	0.00	
200	9.31	5.76	2.20	0.00	
300	9.50	5.85	2.20	0.00	
400	9.61	6.56	3.50	0.00	
500	8.40	6.51	4.61	0.00	
600	9.20	7.45	5.70	0.00	Average temp 32°C
800	10.60	8.50	6.39	0.00	Tyre pressure 80 psi
900	11.00	8.15	5.30	0.00	Average speed 7.2km/hr
1000	11.50	9.40	7.29	0.00	Gross vehicle weight
1100	12.10	10.00	7.90	0.00	= 13,600 kgs
1200	14.60	13.25	11.90	0.00	(4 wheel truck)
1300	15.21	13.11	11.00	0.00	
1400	12.60	10.95	10.30	0.00	
1500	13.30	11.70	10.09	0.00	
1600	13.70	12.15	10.60	0.00	
1800	14.20	10.90	9.60	0.00	

ARD supervision without the use of heavy equipment. The total length of the 3.5-m-wide pavement is 434 m, and the cost was U.S. $11,100. This is equivalent to U.S. $25,600/km, of which labor costs amounted to U.S. $8,060/km. A summary of the construction cost is presented in Table 10.

Performance of ICB Pavements

The performance of interconnecting block pavements has been studied and the results are reported in Tables 7–9. ARD currently use a concrete block 80 mm thick, which provides adequate performance when made with 1:2:4 concrete (by volume). Concrete blocks 50, 80, and 100 mm thick were compared when subjected to repeated loading from a four-wheel vehicle with a gross weight of 13,600 kg. The resultant rutting and cracking presented in Tables 7–9 indicate that performance is directly related to the block thickness.

In northern Thailand, the mountainous terrain, high rainfall, and steep gradients of up to 17 percent combine to cause severe erosion of the surface of unpaved roads. The problem is exacerbated by the poor cohesion and low plasticity of the materials available both in situ and within reasonable haul distance. In some cases, road surfaces are damaged further by trucks using chains on their wheels. The erosion problem is shown in Figure 3, whereas a solution is shown in Figure 4, in which a concrete block pavement has been successfully constructed on a road in hilly terrain and with a steep gradient.

TABLE 10 SUMMARY OF CONSTRUCTION COSTS OF THE FIRST ICB PAVEMENT IN THAILAND

No.	Type of work	Unit	Quantity	Material cost US $	Labour cost US $
1	Scarify and compact existing laterite surface (>100mm depth)	M^2	1,953	970	16
2	Lay sand bedding	M^2	1,953	251	493
3	Lay ICBs and compact	M^2	1,432	84	678
4	Manufacture blocks				
	4.1 ICBs	No	42,960	4,438	1,696
	4.2 Kerbs	No	1,752	1,101	276
5	Timber moulds	M^3	70	553	118
6	Construct shoulders	M^3	46	197	158
7	Install sand drains	No	868	6	63
			Total Cost =	7,600	3,498
			Equivalent to US $25,578/km		

Note: Timber moulds were used only in the first ICB project. Steel moulds are currently used and cost US $3 each.

FIGURE 3 Erosion on steep gradient.

FIGURE 4 Concrete block pavement on steep gradient.

It is the intended policy of the ARD to continue using this technique to reduce the high cost of maintenance on these roads in this type of terrain.

The ICB pavement can also solve the problem of serious differential settlement of rural roads in swampy areas because of the lower cost of maintenance and the capacity to support heavy loads. The first project constructed in a swampy area was at Petchaburi, Thailand. The total length of the ICB pavement was 1.20 km, and cost U.S. $38,700, or U.S. $32,250/km.

A total of 215,040 blocks and 4,880 kerbs were used in the project. The road has an ADT of over 1,000 vpd including 60 heavy trucks (of 20 to 25 tonnes gross weight) and 980 medium trucks, pick-ups, and cars as shown in Figure 5. During construction, some sections exhibited large ruts caused by the heavy trucks, but after recompaction of the subgrade and subbase and relaying of the concrete blocks the road remains in good condition after 2 years' service as shown in Figure 6.

Construction Method

The method used to construct ICB paving was as follows:

1. ARD manufactured and initially provided timber moulds for the projects. In current projects, steel molds with three-block capacity are used.
2. ARD located the block manufacturing site and stockpile of materials near the construction site to minimize haulage costs.
3. Demonstrations of construction and laying of concrete blocks were given by 10 skilled laborers of the ARD training center.
4. Laborers were organized into groups performing specific tasks. These consisted of
   a. A concrete mixing group of 6 laborers.

FIGURE 5 Heavy traffic on block pavement.

FIGURE 6 Concrete blocks after 18 months' trafficking.

b. A concrete pouring group of 20 laborers provided with 55 sets of timber molds, each set with a capacity of six blocks. Each mold set could be used three times a day and the daily total production rate was approximately 1,000 blocks.

5. After pouring, each block was cured for 28 days. Twenty laborers were used to excavate and install concrete kerbs concurrently with the placement of the sand bed. Laborers were then used to lay the blocks, each individual laborer having an output of 10 to 30 m^2 per day, while four laborers were normally used to fill the joints between blocks with sand. Final treatment was the compaction of the shoulders and the construction of drainage holes.

The manufacturer of kerbs and blocks can be done using either timber or steel molds. ARD uses timber molds for kerbs, which are larger than blocks, because they are lighter than steel, but only steel molds are used for interlocking blocks. Cast-in-place kerbs give the best results in terms of quality and cost. The 1:2:4 concrete mix provides a specified cube strength for the concrete blocks of 240 kg/cm^2.

In all cases, the ICB paving is laid on a subgrade of at least 6 percent CBR with the natural material being improved to exceed this figure when necessary. A laterite subbase at least 100 mm thick is necessary although the actual thickness provided will be dependent on the predicted wheel loads and ADT. Finally, a sand layer 50 mm thick is provided on which the blocks are laid, as shown in Figure 7.

The joints between the blocks are filled with sand and dry cement proportioned 1:1, which is compacted using vibrating plate compactors to completely fill the joints and to ensure complete interlock and load transfer between adjacent blocks. A completed pavement is shown in Figure 8. Figure 9 shows the current block and mould design, and Figure 10 shows typical construction details.

CONCLUSIONS

The ARD/TRRL collaborative research study demonstrates the use of appropriate equipment for carrying out the surface maintenance of gravel roads. In many developing countries, there is a scarcity of foreign exchange that would be required to purchase new or replacement machinery. It is far more appropriate to use machinery that is not overly complex and can be maintained and repaired using local expertise and materials. The main criteria should be whether the equipment can achieve standards similar to those obtained by the more complex machinery with the same degree of reliability.

The most important finding so far from the study is the potential savings in maintenance costs by using the tractor-

FIGURE 7 Laying concrete blocks.

FIGURE 8 Completed lane of concrete blocks.

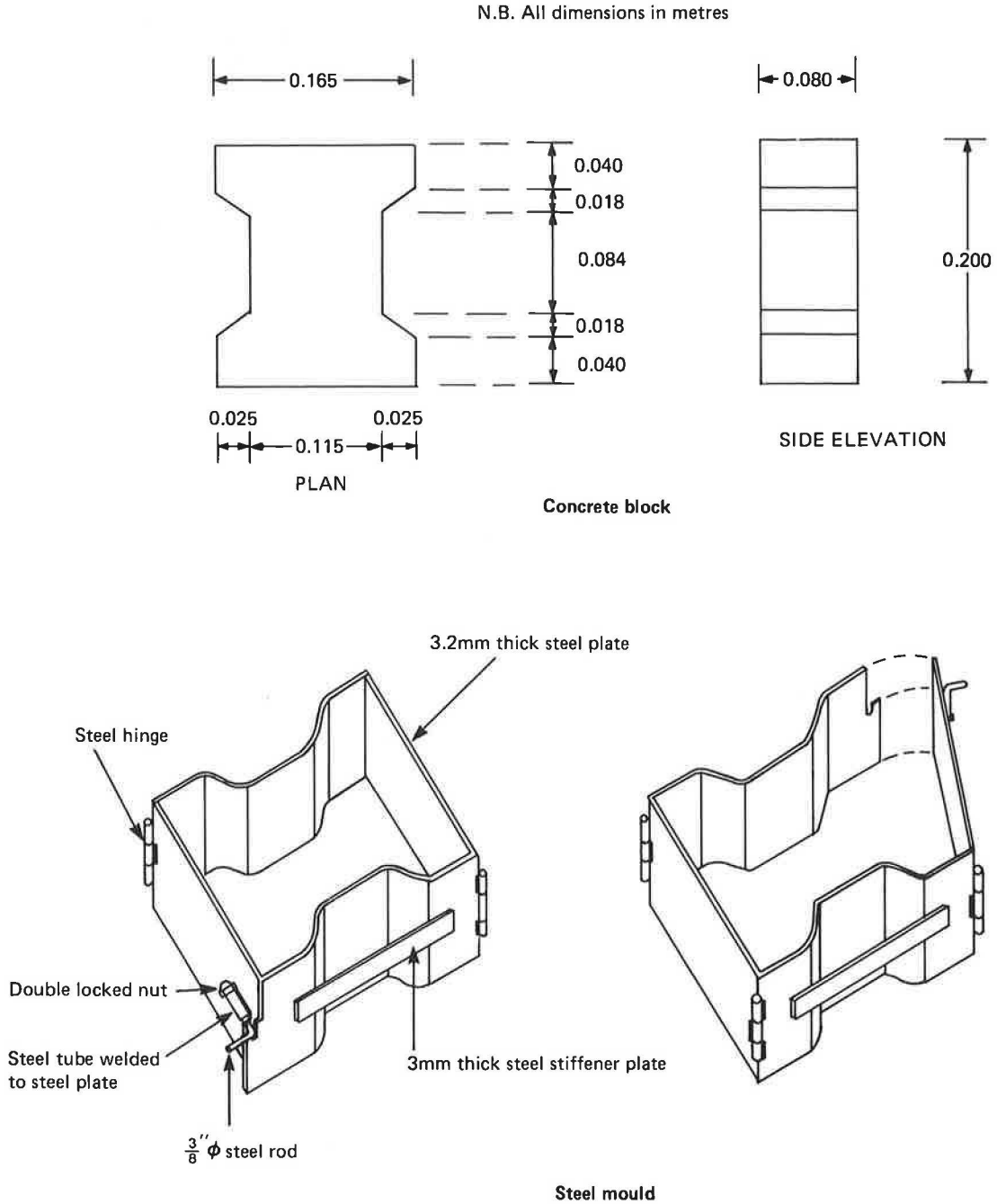

FIGURE 9 Concrete block and steel mold specification.

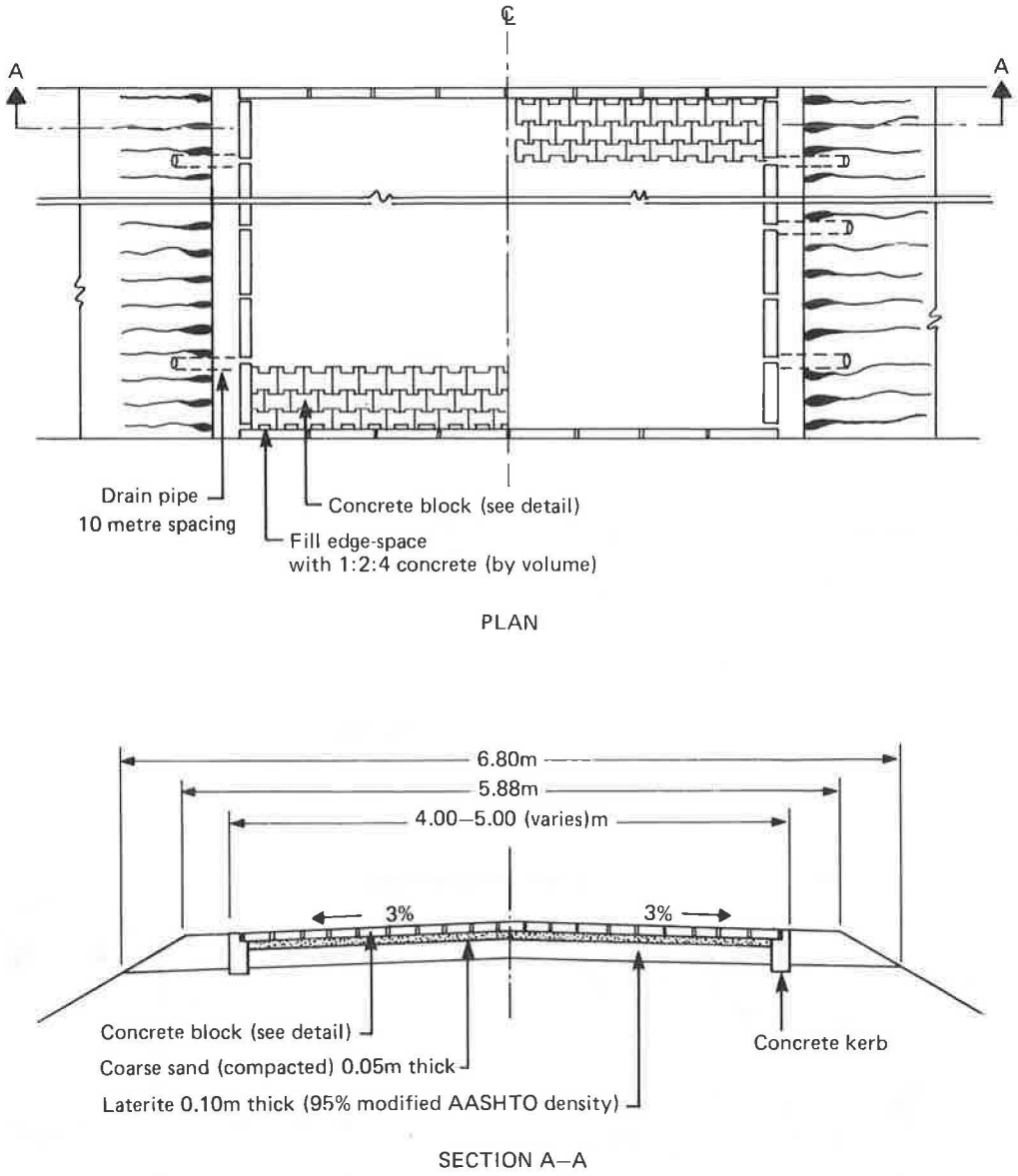

FIGURE 10 Method of construction of interlocking concrete block pavement.

towed grader. If these savings are sustained, then it will afford ARD the opportunity to increase maintenance on the higher trafficked roads to reduce road user costs. Results from this research will also help the ARD to optimize the frequency of grading for roads with different levels of traffic, materials, and climate.

The use of concrete blocks to reduce maintenance costs in specific areas such as steep terrain and swampy ground has been demonstrated to provide a cost-effective alternative maintenance option.

ACKNOWLEDGMENTS

This research forms part of the research program of the Overseas Unit of the British Transport and Road Research Laboratory (Unit Head J. S. Yerrell) and is published by permission of the Director, Transport and Road Research Laboratory, and reproduced by permission of Her Brittanic Majesty's Stationery Office.

Due acknowledgement must be given to Chodok Virathum Poonsawat and Chaiwat Arunotivivat of the ARD and their staff for the unstinted support and enthusiasm given to the project.

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Views expressed herein are not necessarily those of the British Government.

Program Prioritization Strategies for the National Park Road System

JEFF RAPOL

An application is described resulting from a coordinated direction of resources and labor that effectively demonstrates the benefits of applied highway research to funding strategies for low-volume road networks. The National Park Road and Parkway System comprises more than 8,500 mi, most of which is low-volume roads. At current funding levels, the backlog of needs has increased, and with the present budget climate further deterioration of this national recreational resource may be inescapable. The policies developed for the administration of the park roads and parkways have resulted in the creation and regular updating of several data bases containing inventory data, traffic and accident data, repair and replacement costs, and limited maintenance records for road sections. The information in these data bases was used to develop performance models and subsequent funding strategies. The planning methodology is used for data collection, inventory, and rating process; generating performance models and lookup tables; incorporating needs based on design class; predicting road segment condition; and prioritizing road repairs system-wide on the basis of funding constraints for discrete funding cycles. The dBASE III Plus (TM) programs were developed to show the present condition and needs of a road segment, predict the condition and needs at some future time, and choose those segments that would most effectively use available funds. The condition and needs of each segment are then forecasted to the next funding cycle. The condition and needs analyses are then repeated using the anticipated funding for that funding cycle. This iterative process continues for a 20- to 25-year period in six cycles. The model can be used by all management levels for planning, administration, maintenance and system management, and program budgeting.

An effective planning strategy is described for choosing road repair sections for a network consisting of about 5,000 mi of predominantly low-volume paved roads. The planning strategy uses a performance prediction model, an improvement cost table, and pavement condition information gathered by the FHWA for the National Park Service (NPS) as part of the Federal Land Highway Program. The three major features of the planning strategy are as follows:

1. The performance prediction model used to characterize the paved roads on the network.
2. The cost improvement table used to determine repair costs on the basis of the National Park Service road classification.
3. The analysis program used to forecast condition and repair needs on the basis of the performance curve and improvement cost table.

Federal Lands Highway Program Office, FHWA, Washington, D.C. 20590.

PERFORMANCE PREDICTION MODEL DEVELOPMENT

The performance prediction model used to forecast pavement condition over time was developed by the Eastern Federal Lands Highway Division office of FHWA during 1988 and 1989. The data for the model are part of the Road Inspection Program data bank maintained by the Federal Lands Highway Office. This data bank contains the results of three periodic road inspections dating back to 1978. It has about 8,300 records, of which 4,500 describe the attributes of public-use paved roads.

The performance model was developed in two stages. First, performance models were generated for each of the 10 National Park Service regions. The data for these models came from road sections with the following known attributes:

- Average daily traffic (ADT),
- Construction history,
- Original construction date,
- Only routine maintenance performed on the section since the original construction date, and
- Available data from three rating cycles.

The construction and maintenance history for many of the roads are not as complete as desired. The ADT data and pavement condition data from the Road Inventory Program are more complete.

As part of the Road Inventory Program, each paved road is divided into sections. Each section is rated. The rating procedure evaluates the pavement structure, shoulders, drainage, foundation, and roughness attributes of the road on a 1 to 5 (lowest to highest) scale. The ratings are combined to arrive at a condition sufficiency rating (CSR) for each section on a 30 to 100 scale. New pavements, assumed to have no damage, have a sufficiency rating of 100. Pavements at the lowest tolerable operating condition have a sufficiency rating of 30. The rating criteria and entire rating process used to determine section attributes are described in the *Procedure Manual for the NPS Road Inventory Program* (*1*).

For the model development, the data were fit to a standard parabola symmetric about a line parallel to the x-axis (*2*). The assumption is that the overall trend in park road deterioration is irreversible. Artificial terminal points, based on this rating procedure, are chosen on the y-axis at 100 for new pavements and at 30 for the pavements at the lowest tolerable operating condition. During the curve-fitting process (*3*), some adjustments to the data were necessary because condition ratings increased from one cycle to the next. For example, a pave-

ment section rating could increase from one cycle to the next if improvements to drainage or shoulder areas were performed between rating cycles. This could occur, while the pavement surface condition deteriorated during the same period. In these cases, the previous drainage and shoulder condition ratings were substituted and the pavement condition was recalculated.

The performance curves showed varying pavement life (the time from 100 to 30 CSR) for each region. A statistical test of the accuracy of the performance curves for each region gave unacceptable results. This led to the second stage in the development of the performance model.

The second stage in the model development involved comparing the region curves and derived a model that would represent the primary paved road network. The region data were reanalyzed and combined to derive a model for overall system planning. The data used for the system model development are summarized in the following table, which presents the number of routes, sections, and miles of road that had the most complete information available in the data bank and also satisfied the criteria.

NPS Region	*Number of Parks*	*Number of Routes*	*Number of Sections*	*Number of Miles*
Alaska		no data used		
Mid-Atlantic	2	7	12	10
Midwest		no data used		
National Capital		no data used		
North Atlantic	2	9	15	10
Pacific Northwest		no data used		
Rocky Mountain	11	15	19	67
Southeast	4	5	47	65
Southwest	3	5	7	36
Western	4	10	12	27
Total	26	51	112	215

The performance model shown in Figure 1 gives a representation of pavement service life for the paved park roads and parkways on a system-wide basis.

The equation for the model is

$$X = 29.6 - 0.00604(Y - 30)^2$$

where X = pavement age and Y = condition rating.

The model shows the four stages of pavement condition that are used to characterize the park road and parkways. Good pavements have a CSR value from 85 to 100. Fair pavements have a CSR value between 65 and 85. Poor pavements have a CSR between 45 and 65. Failed pavements have a CSR rating below 45. On the average, and for this overall planning strategy, the model shows that a park road has a service life of about 28 years. The ages at which the pavement drops from good to fair (11 years), fair to poor (23 years), and poor to failed (28 years) are used as trigger points in the analysis, described later. The condition ranges that correspond to these ages are used in the development of the improvement cost table.

IMPROVEMENT COST TABLE DEVELOPMENT

The improvement cost table used in forecasting pavement needs is a sliding scale cost for heavy maintenance or overlay, rehabilitation, and construction, based on the pavement CSR value. Only routine maintenance costs (no highway trust funds) are associated with a CSR above 85. In general, this amount averages \$10,000/mi to \$20,000/mi, depending on functional classification. Heavy maintenance or overlay costs are applied to road sections that have a CSR value of 65 to 85. Rehabilitation costs are applied to road sections that have a CSR value of 45 to 65. Reconstruction costs are applied to road sections that have a CSR value less than 45. Table 1 presents the improvement costs used in the analysis program.

Table 1 was developed using historical cost data as compiled by the Eastern, Central, and Western Federal Lands Highway Division Offices. The values in the table are based on the average cost to improve park roads and parkways. The values include basic shoulder costs, average roadside improvement

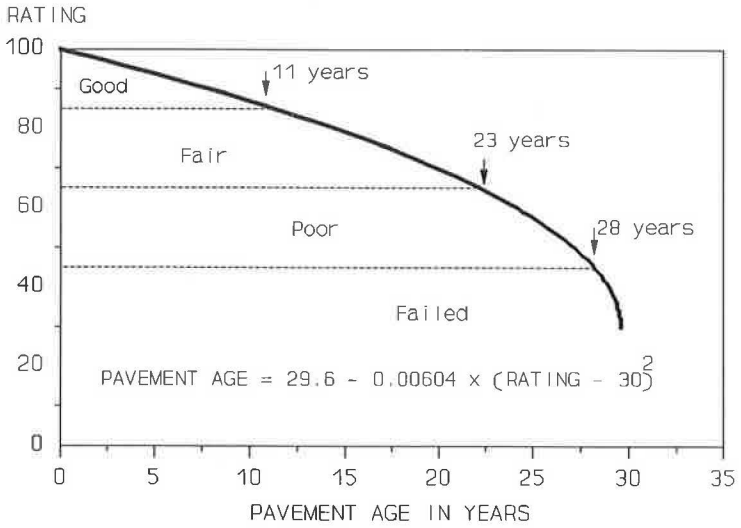

FIGURE 1 Performance model for public use primary paved roads.

TABLE 1 MAINTENANCE AND CONSTRUCTION COST PER MILE FOR PRIMARY PAVED ROADS

	CONDITION RATING	CLASS I & II	CLASS III & OTHERS	CLASS VII
MAINTENANCE COSTS	85-100	$10,000	$10,000	$20,000
CONSTRUCTION COSTS				
REPAIR	65-85	$200,000	$130,000	$400,000
REHABILITATE	45-65	$500,000	$420,000	$1,000,000
RECONSTRUCT	< 45	$950,000	$820,000	$1,950,000

costs, engineering costs, and related National Park Service costs. The costs per mile, rounded to the nearest $10,000 and broken down by National Park Service Road Classification, are given in the table.

The *Park Road Standards* (*4*) definitions for road class are as follows:

Class	*Definition*
I	Principal Park Road/Rural Parkway. Roads that constitute the main access route, circulatory, tour, or thoroughfare for park visitors.
II	Connector Park Road. Roads that provide access with a park to area of scenic, scientific, recreational or cultural interest, such as overlooks, campground, etc.
III	Special Purpose Park Road. Roads that provide circulation within public-use areas. These roads generally serve low-speed traffic and are often designed for one-way circulation.
VII	Urban Parkway. These facilities serve high volumes of park and non-park-related traffic and are restricted, limited-access facilities in an urban area.

"Others" indicates roads that fall into Class IV, V, or VI. These road classes are similar in design to Class III roads.

All costs associated with Class I, II, and III & Others are based on a two-lane facility. All costs associated with Class VII roads are based on a four-lane facility. The cost differences between Class I and II, Class III & Others, and Class VII are related to the number of lanes and the amount and type of traffic loadings used in the design standards for each road class. Class I, II, and VII roads are low- to high-volume roads with frequent bus and light truck traffic. Class III & Others have low-volume traffic with only occasional buses.

These costs were used in conjunction with the performance model to develop the program that forecasts needs for the system.

PAVEMENT NEEDS FORECASTING PROGRAM

The needs forecasting program was written using dBASE III+ programming language. It has several procedures that use the performance model and the improvement cost table, along with other input variables, to predict future needs and condition of paved-road sections. The forecasting procedure orders by priority the road sections on the basis of vehicle miles of travel (VMT) and pavement CSR. The priority scheme is adaptable to other criteria.

The program is initialized by two data input menus. The first menu prompts the user for an area of interest or scope, which can be an individual park, an NPS region, an FHWA division, or the entire network. The analysis program using the entire NPS road network as the scope will be described later.

The second menu prompts the user for information concerning authorization levels (currently at $60 million/year), discount factor, whether or not to include roadside appurtenances, an upper limit on the CSR value above which no repairs are necessary (for park roads and parkways, a CSR value above 85 was considered to require no highway trust funds), and the percent funding cap desired for road sections in failed condition. Another funding split is possible for sections that will fall into the next lower condition category in 5 years.

Example parameters follow:

Scope:	All public use paved roads
Funding cap for failed roads:	30 percent
Available funds per year:	$105 million
Discount factor (per yr):	2 percent
Upper limit for no funds needed:	CSR above 85
Funding split for roads dropping to lower category in five years:	75 percent

Program Flow Using All Public Use Paved Road Sections

All records are added to a temporary data base. The program makes the first pass through the data and updates the condition and needs to FY 1989 on the basis of the performance model and cost table previously described. The initial age of the pavement is determined from the CSR value and inspection dates given in the data base.

The program then normalizes this age and condition to 1989. Afterwards, the program consults lookup tables to assign corresponding CSR values and needs for that age. For instance, if a Class I section was rated in 1985 and had a CSR value of 88 at that time, it was equivalent to a 9-year-old road with no major repair needs. However, in projecting the pavement to 1989, the pavement is now 14 years old, has a projected CSR value of 81, and has needs associated with Class I fair roads ($200,000/mi repair costs per two-lane facility). The needs are calculated using length and number of lanes for the section as follows:

$$\text{Needs} = \text{Length} \times \text{Number of Lanes} \times \text{Appropriate Cost/lane-mile}$$

The fields from the data base that have information essential to this analysis are as follows:

AGE_BASE	The difference between the inspection year and 1989 rounded to the nearest year. For example, if a section was inspected in March 1987, the

AGE_BASE would be 2 years. Similarly, if the section was inspected during August 1987, the AGE_BASE would be 1 year.

PARKCODE — Approved number corresponding to an NPS park.

LGTH — Length of road section.

AGEFIX — The age of the section corresponding to the CSR value in the main data base as calculated using the following performance model:

$$AGE = 29.6 - 0.00604 \times (CSR - 30)^2$$

where AGE = section age, CSR = sufficiency rating at last inspection.

COUNT — Each section has a count of one.

VMT_RANGE — The main database is sorted in decreasing order of VMT. Furthermore each section is assigned to a VMT range according to the number of vehicles per day (vpd) as follows:

Annual VMT (vpd)	*VMT_RANGE*
>8000	1
4000–8000	2
1000–4000	3
400–1000	4
200–400	5
<200	6

This information is used to generate a report on needs by VMT category.

DES_RANGE — Each section is assigned a range according to road class.

NPSREGION — The National Park Service region in which the section is located.

FHWAREGION — The Federal Lands Highway division in which the section is located.

FIX_CLASS — Number of paved lanes.

EQUATION — This field provides a mechanism to use separate performance curves for each road section, route, region, or division.

Priority Assignment

The program uses AGE_BASE to calculate the projected CSR value for each section at the beginning of the analysis. It then assigns CSR_RANGE and a repair priority (PRIORITY) to each section on the basis of the following assignment tables:

CSR_Range Assignment Table:

CSR	*CSR_Range*	*Condition*
85–100	1	GOOD
65–85	2	FAIR
45–65	3	POOR
<45	4	FAILED

Priority Assignment Table:

Condition at Beginning of Cycle/ Condition 5 Years Later	*Relative Priority*
GOOD/FAIR	1
FAIR/POOR	2
POOR/FAILED	3
FAILED	4
POOR/POOR	5
FAIR/FAIR	6
GOOD/GOOD	7

Needs are assigned to each section, by road class, according to an improvement cost table. The data base is then sorted from highest to lowest ADT value by relative priority.

Next, the program creates an internal table summing the needs according to the priority assignment table and determines the relative percent needs for each priority category.

Condition Now/and in 5 Years	*Relative Priority*	*Needs (%)*
GOOD/FAIR	1	GF
FAIR/POOR	2	FP
POOR/FAILED	3	PFD
FAILED	4	FD
POOR/POOR	5	PP
FAIR/FAIR	6	FF
GOOD/GOOD	7	GG

These percentages (total needs of 100 percent) establish the relative needs of the system according to condition rating, and the four condition categories are subdivided to establish seven priority categories.

The pavements in the GOOD range have been subdivided into two categories, GOOD/GOOD and GOOD/FAIR. The sections in the GOOD/GOOD category are from 0 to 6 years old and have no repair costs associated with them. The GOOD/FAIR category contains sections that are 7 to 11 years old, have no cost associated with them at the beginning of the funding cycle, but are forecasted to fall into the FAIR category in 5 years. Routine maintenance of these sections could prevent them from dropping into the FAIR category in 5 years.

The pavements in the FAIR range have been subdivided into two categories, FAIR/FAIR and FAIR/POOR. The FAIR/FAIR category contains sections that are 11 to 17 years old and the FAIR/POOR category contains sections that are 18 to 23 years old and will likely be in the POOR range in 5 years. Funds devoted to repairing these sections could prevent them from falling into the POOR category. This would save considerable funds for other sections because the cost of repairing POOR sections is about 2.5 times the cost of repairing FAIR sections.

The pavements in the POOR range have been subdivided into two categories, POOR/POOR and POOR/FAILED. The POOR/POOR category contains sections that are 23 to 24 years old and the POOR/FAILED category contains sections that are 24 to 28 years old and will likely be in the FAILED range in 5 years. Funds devoted to repairing these sections could prevent them from falling into the FAILED category. This would save funds because repairing FAILED sections is about twice the cost of repairing POOR sections.

Funding Distribution

Having established needs and priorities, the program takes the available funding and divides it into four funding accounts. The accounts are dedicated to good, fair, poor, or failed road sections according to the percent needs determined for each condition range. In the case of good, fair, and poor roads, the program uses the input parameter "Funding split for roads dropping into a lower condition category in five years" to further divide these accounts. This procedure makes more funds available for roads that would fall to a lower category during the next 5 years. The funding accounts are as follows:

Funding Account	*Condition Category*
1	Good condition for at least 5 years
1	Good dropping to fair condition in the next 5 years
2	Fair condition for at least 5 years
2	Fair dropping to poor condition in the next 5 years
3	Poor condition for at least 5 years
3	Poor dropping to failed condition in the next 5 years
4	Failed roads

In this example, Figure 2 shows the overall characteristics of the road network at the beginning of FY 1992.

The program uses the information in the preceding table as a starting point. It distributes available funding to the network automatically according to the percent needs of each condition range. In the above example, 22 percent of the available funds would be distributed to roads in the FAIR range, 29 percent to roads in the POOR range, and 49 percent to roads in the FAILED range. However, the input parameter limited the percent funding in the FAILED range to 30 percent. The program takes the extra 19 percent of the funds and redistributes them to the FAIR and POOR ranges, again according to relative needs. Figure 3 shows how the program would distribute $105 million per year each year for 4 years using the planning strategy.

Figure 4 shows the percent of available funds that were distributed to each condition range. The funds are discounted using the discount factor given in the input parameters. The number of sections repaired and the total mileage repaired are also given.

The program prepares a summary of lengths, weighted condition ratings, and needs for the before and after conditions of the network similar to Table 2.

Table 2 presents the class, condition, and needs of the system at the beginning of FY 1992. It was derived using the anticipated funding for FY 1990 and 1991.

Table 3 presents the updated class, condition, and needs of the system at the beginning of FY 1996. It was derived using the $105 million/year for FY 1992 through 1995. It shows an overall improvement in the network.

Funding Cycles

The process of reevaluating each section at the beginning and end of each funding cycle, assigning PRIORITY, and sorting the data base according to ADT is carried out for the following funding cycles:

PRE FY 1990 (projected CSR, needs, and FY 1989 funding)
FY 1990 through FY 1991 (end of Highway Act of 1987)
FY 1992 through FY 1996 (presumed Highway Act of 1992)
FY 1996 through FY 2000 (presumed Highway Act of 1996)
FY 2000 through FY 2004 (presumed Highway Act of 2000)
FY 2004 through FY 2008 (presumed Highway Act of 2004)
FY 2008 through FY 2012 (presumed Highway Act of 2008)

Files are generated after each pass through the data with the temporary data base containing the final results of sections funded throughout the analysis. This data base can be copied to a permanent file for further analysis and possible use as a project selection list.

```
1992 - NPS NETWORK                    NEEDS  %NEEDS   MILES %MILES
NEEDS ($1000) = 1,566,869  G              0     0%   1,544    32%
       LENGTH =     4,856  F        344,209    22%   1,613    33%
 WEIGHTED CSR =        72  P        451,934    29%     872    18%
                           FD       770,726    49%     827    17%
                         P+VP     1,222,660    78%   1,700    35%
```

FIGURE 2 Characteristics of road network, beginning of FY 1992.

```
AVAILABLE     NEEDS    LENGTH    COND(FY-1992) - COND(5 YEARS LATER)
        0         0       853    GOOD (G)        GOOD
        0         0       691    GOOD            FAIR
    7,944   256,082     1,220    FAIR (F)        FAIR
   23,833    88,127       393    FAIR            POOR
   10,431   322,164       625    POOR (P)        POOR
   31,292   129,770       247    POOR            FAILED
   31,500   770,726       827    FAILED (FD)
```

FIGURE 3 Distribution of $105 million/year for 4 years for repairs.

```
THE 'DISCOUNTED FUNDS' WERE DISTRIBUTED ACCORDING TO ROAD CONDITION*
   CONDITION   ACTUAL SPENT ($1000)      PERCENT
    GOOD (GF)              0 (      0)       0.0
    FAIR (FP)        116,293 ( 87,222)       30.3
    POOR (PFD)       152,679 (114,509)       39.7
    FAILED           115,296                 30.0
    TOTAL            384,268                      18   FUNDS LEFT
    593 SECTIONS REPAIRED DURING THE LAST CYCLE, TOTALING  876 MILES.
 *FUNDING WAS RE-DISTRIBUTED WHEN AVAILABLE FUNDING EXCEEDED NEEDS.
```

FIGURE 4 Distribution of funds by condition range.

TABLE 2 SUMMARY MATRIX FOR PUBLIC USE PAVED ROADS, FISCAL YEAR 1992

CONDITION		DESIGN CLASS I	II	III+OTH	VII	TOTALS
CSR > 85 (GOOD)	NEEDS*	0	0	0	0	0
	LENGTH	1,206	143	116	80	1,544
	CSR	95	95	93	97	95
CSR 65-85 (FAIR)	NEEDS*	237,971	49,825	29,109	27,304	344,209
	LENGTH	1,173	196	199	45	1,613
	CSR	80	76	76	76	79
CSR 45-65 (POOR)	NEEDS*	300,031	90,540	60,017	1,346	451,934
	LENGTH	599	144	126	3	872
	CSR	58	56	56	60	57
CSR < 45 (FAILED)	NEEDS*	375,304	258,347	136,941	134	770,726
	LENGTH	392	254	181	0	827
	CSR	31	31	31	30	31
TOTALS	NEEDS*	913,306	398,712	226,067	28,784	
	LENGTH	3,369	737	623	127	
	CSR	76	60	62	89	

*$1000 NEEDS ($1000) = 1,566,869 NPS NETWORK
LENGTH = 4,856 TOTALS - FY-1992
WEIGHTED CSR = 72

TABLE 3 SUMMARY MATRIX FOR PUBLIC USE PAVED ROADS AT $105 MILLION/YEAR, FISCAL YEAR 1996

CONDITION		DESIGN CLASS I	II	III+OTH	VII	TOTALS
CSR > 85 (GOOD)	NEEDS*	0	0	0	0	0
	LENGTH	1,667	302	183	110	2,261
	CSR	94	96	94	94	94
CSR 65-85 (FAIR)	NEEDS*	206,627	25,630	21,745	6,805	260,807
	LENGTH	1,012	120	151	15	1,297
	CSR	76	75	74	73	76
CSR 45-65 (POOR)	NEEDS*	113,123	17,398	19,294	80	149,895
	LENGTH	227	31	43	0	300
	CSR	50	49	49	51	50
CSR < 45 (FAILED)	NEEDS*	445,251	305,953	193,049	1,774	946,027
	LENGTH	464	285	246	2	997
	CSR	33	32	32	39	32
TOTALS	NEEDS*	765,001	348,981	234,088	8,659	
	LENGTH	3,369	737	623	127	
	CSR	77	66	62	91	

*$1000 NEEDS ($1000) = 1,356,729 NPS NETWORK
LENGTH = 4,856 TOTALS - FY-1996
WEIGHTED CSR = 74

Automatic Funding Redistribution

The program uses funds from each account until they are essentially exhausted. There may be a few thousand dollars left in an account because of section repair costs exceeding the amount left in the account. These funds are automatically added to the failed road funding account. For example, say the funding account for fair roads has $10 million in it, funds are depleted (section repaired) until $6,000 is left in the account, and the lowest repair costs for an unrepaired section in fair condition is $10,000. The program would add $6,000 to the failed road funding account because repair costs of $10,000 exceed the $6,000 left in the fair roads funding account. This automatic funding redistribution is insignificant at the current $60 million/year funding level. However, at higher funding levels the redistribution effectively overrides the funding cap on failed roads because the available funding could exceed the percent needs of the fair and poor ranges.

Funding Priority and Funding Cap on Failed Roads

The analysis process is driven by the percent needs within each road condition category. This funding strategy seeks to balance resurfacing, rehabilitation, and reconstruction projects at all funding levels and to reduce the number of poor

and failed roads, especially when funding levels are higher than the current levels. The needs are determined at the beginning of each funding cycle for fair, poor, and failed roads. If the needs of any category grow at an unusual rate (rapid deterioration because of traffic loading changes, construction procedures, or environmental causes), the strategy compensates by allocating more funds to that category. However, a funding cap on failed roads is necessary for the strategy to work. At current and anticipated funding levels, the number of roads in the failed category would soon cause all funds to be spent on reconstruction projects. The current needs for failed roads is about $700 million and they are projected to be close to about $770 million in 1992. It is unlikely that $770 million will be available at that time for park roads and parkways. Even if it were appropriated, the public would not likely tolerate closing so many park roads at one time, and the possibility of increased inspection and laboratory staffing needs for additional reconstruction projects would have to be addressed. The analysis program can show that the overall system condition deteriorates at a faster rate as more than about 35 percent of funds are dedicated to failed roads, unless the funding level is over $160 million/year.

As a safeguard, the program automatically overrides the cap as funding exceeds needs in the other condition categories.

At anticipated funding levels, devoting no more than 35 percent to failed pavements improves the overall system condition and keeps the number of failed roads near current levels. This is shown in the following example. The comparison of needs (rounded) using authorizations of $60 and $120 million/year with 35 and 50 percent caps on funding for failed pavements shows that devoting a large share of funding to failed roads increases system needs.

	FY 2000 Needs at $60M ($)	*FY 2000 Needs at $120M ($)*
25% cap	2,168,000	1,647,00
35% cap	2,120,000	1,561,000
50% cap	2,176,000	1,682,000

The table shows increased needs at both $60 and $120 million/year funding as more than 35 percent of funds are devoted to failed roads. The result of allowing failed road needs to drive a repair program is even more dramatic after 20 years.

	FY 2012 Needs at $60M ($)	*FY 2012 Needs at $120M ($)*
25% cap	2,773,000	1,370,000
35% cap	2,754,000	1,235,000
50% cap	2,938,000	1,652,000

The table shows that even at double the current authorization level, a funding cap on failed roads is necessary when assigning repair priorities. Allowing failed road needs to drive a program can increase the backlog of needs at a rapid rate.

Realistic Funding Combination at Higher Funding Levels

The analysis program can show that in order to maintain the existing public-use paved roads in roughly the same condition for the next 20 years requires about double the current funding level, with about 30 to 35 percent going to failed sections and 70 to 80 percent of various funding accounts going towards catching sections before they fall to a lower condition category. A statistical check on the accuracy of the analysis program should be possible as additional inspection cycles are analyzed.

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Bridges and Culverts

Timber Bridges: Background, Attributes, National Direction, and Stressed Timber

CLYDE G. WELLER AND JOHN CRIST

The interest in, and momentum for use of, timber bridges has been growing in the last few years and is expected to mushroom in the near future. This trend effectively reverses the declining trend of the last four decades. Why all the sudden interest? There are four broad reasons, termed driving forces, that are generating the interest. These driving forces are (a) economic development (especially rural), (b) awareness, (c) need, and (d) the new stressed timber concept. All of these driving forces are examined and the concept of stressed timber for bridges is expanded. The U.S. Congress appropriated $3.35 million to continue research on stressed timber bridges and construct 80 demonstration bridges across the nation in 1989. That funding level was maintained in 1990. Stressed timber is catching the eye of innovative bridge engineers across the United States. The progress of stressed timber and the potential for its increased use and more innovation are greater than ever. Stressed timber is not new; in Ontario, Canada, stressed-timber bridges are more than 10 years old and show outstanding performance characteristics. The process of approving AASHTO standards for stressed-timber bridges has begun.

Historically, timber played an important role in the road, highway, and railroad structures of the United States. The trend decreased in the 1940s when concrete and steel became engineers' materials of choice. Several recent events have brought attention back to timber and its inherent attributes.

Most of the weaknesses of timber have been solved. The potential life of pressure-treated timber bridges easily competes with steel and concrete.

The driving forces that have brought attention back to timber, the opportunities available to the timber industry, the attributes of timber (wood), the timber bridge initiative, and the new concept in timber design—stressed timber—are discussed.

DRIVING FORCES

The driving forces that are generating the revived interest in timber bridges are (a) economic development (especially rural), (b) new awareness, (c) need, and (d) the new stressed timber design concept.

Economic development, particularly rural development and diversity, is much in the mind of the public and its elected representatives. Timber bridges offer an opportunity to use abundant, underutilized, local species and capture the value added through all phases, from stump to bridge, of harvesting, milling, fabrication, and construction. Coupled with this opportunity for increased employment, the installed bridges upgrade the transportation infrastructure, thereby making rural areas more accessible for further economic development. The new stressed timber concept has allowed the use of most local species and local fabrication.

The public also has increased awareness of the properties of wood as a material and its performance when used in engineered structures such as bridges. Wood is such a common material that it has been taken for granted in the past. The general public has a growing awareness, albeit maybe still small, of the attributes of wood and the possibilities of using it to upgrade the deficient transportation system. The public has also been made aware of deficient bridges through the national media's dramatic coverage of bridge collapses, including loss of life.

The awareness of our deficient bridges leads to the third driving force—need. The nation, according to the inventory by the FHWA, has 575,000 bridges. Of these, 242,000 (42 percent) are structurally deficient or functionally obsolete, and are in need of rehabilitation or replacement. Of the nation's bridge inventory, currently 12.6 percent of all bridges are constructed of timber and that percentage has been declining. Timber bridges are predominantly located in the off-federal-aid system and are located on rural and secondary roads. The total number of bridges on rural and secondary roads is 470,000. Of these, 414,000 (88 percent) are 40 ft or less in length. Bridges of this length are ideally suited for timber construction.

The last and main driving force in making timber more attractive is the new concept in design—stressed timber. This topic will be discussed in detail.

OPPORTUNITY

The national need to upgrade deficient bridges can be turned into a greater opportunity for local industry in forest products and stump to participate in bridge fabrication. Stressed timber allows for the use of many underutilized local species.

An average size timber bridge contains approximately 10,000 board feet (bf) in the deck and superstructure. If timber is used for the abutments or substructure, an additional 10,000 bf will be used. Multiplying the amount of timber used in a typical bridge by the number of deficient bridges gives a gross approximation of the maximum potential market of 5 billion bf. Recognizing that all deficient bridges will not be replaced, and that all replaced will not be constructed with wood, an optimistic goal might be 20 percent of the maximum potential.

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Probably a bigger opportunity, although more intangible than markets, is favorable visibility. Timber bridges offer the forestry profession the opportunity to greatly increase its cooperators or network by working with individuals and organizations interested in transportation, in the welfare of people, and in rural communities.

ATTRIBUTES OF WOOD

Properties and attributes of wood become advantages when used in modern timber bridge systems. Wood is easy to fabricate using ordinary hand and power tools that are available in local municipal highway shops. Local crews can do much of the fabrication with cost savings to a local government or owner.

Wood also has a high ratio of strength to weight. In bridge rehabilitations, deteriorated concrete decks can be removed from structurally sound steel or concrete stringers and replaced with timber decks. When old HS–15 designed bridges need upgrading to highway design HS–20 loads, a timber replacement deck may reduce the dead load enough to allow the high live load carrying capacity. Timber weight density is approximately ⅓ that of concrete. The weight savings gained by using wood can allow for use of existing abutments to achieve greater weight limits on existing or rehabilitated structures.

Another major attribute of wood is its immunity to deicing salts. This property is one of the main reasons why timber bridges have long life expectancies and low maintenance costs.

Other properties of wood that contribute to timber bridge attractiveness are the excellent fire performance of large timbers, the high degree of sound and thermal insulation provided, and the high impact resistance. AASHTO requires no reduction in strength for impact on timber bridges. The warmth and beauty of wood are evident in timber bridges, and the slim profile of modern designs are aesthetically pleasing and blend into many rural and local settings.

TIMBER BRIDGE INITIATIVE

In 1989, the U.S. Forest Service funded construction of stressed timber bridges as follows: five in the eastern region, three in the southern region, two in the northern region, and one in the Pacific coast region.

Congress funded $3.35 million to do research, set up a Timber Bridge Information Resource Center, and construct demonstration timber bridges outside of national forest lands. The 1989 Congressional appropriation was divided in major programs as follows:

Program	*Amount ($)*
Cost-share demonstration bridges	2,000,000
Research	650,000
Technology transfer	700,000

Seventy-nine timber bridges, 60 of which use the stressed-timber concept, were selected and constructed in the spring of 1989 through fall of 1990. The bridges ranged from slab decks, through stressed T-sections and boxed girders. Many species of wood were used, both hardwoods (elm, red oak, black oak, yellow poplar, and maple) and softwoods (ponderosa pine, red pine, douglas fir, and lodgepole pine).

The funds for demonstration bridges were increased by $5.5 million through matching funds from local and state governments. These funds allowed bridge projects in 30 states across the nation. The demonstration bridges were picked for visibility. Visibility is often increased by the project being done in conjunction with major conferences. The designs were typically innovative designs. Many will be monitored to develop performance data. Information on performance will be presented to AASHTO to gain approval for these innovative designs. AASHTO approval is necessary if federal funds from the FHWA are to be expended in future installations. Also, many state transportation departments require AASHTO-approved designs if state funds are to be expended for county or local installations.

Again in 1990, Congress funded $3.35 million to do research, continue the technology transfer program through the Morgantown, West Virginia, Timber Bridge Information Center (TBIRC), and construct demonstration timber bridges outside of national forest lands.

TIMBER BRIDGE CONFERENCES

There have been many timber bridge conferences held to date: one in 1986 in Oregon; two in 1987, in Pennsylvania and in Colorado; two in 1988, in West Virginia and in Wisconsin; and several in 1989 in Idaho, Vermont, Michigan, and Alabama, with several more held in 1990. Attendance at each of the conferences was near 150 participants. The Forest Service's northwest region and representatives of industry held training sessions in Oregon, Pennsylvania, Ohio, West Virginia, and Wisconsin in 1988 and 1989. A number of other states and Forest Service regions have requested timber bridge conferences as well, and are planning follow-up technical training sessions.

TRB, AASHTO, AND NFPA TIMBER BRIDGE COMMITTEE PROGRESS

The AASHTO standards exist for rough-sawn and glulam bridges. The Forest Service, in cooperation with the University of Wisconsin and West Virginia University, is currently writing AASHTO specifications for stressed timber. AASHTO is rewriting all bridge specifications into load resistance factor design (LRFD) through TRB and NCHRP. The NCHRP Timber Bridge Technical Committee has been commissioned to write the LRFD specifications within 3 years. A rough draft of those specifications was completed in 1990. The stressed-timber concept was included in that writing.

The AASHTO Timber Bridge Technical Committee had excellent meetings in 1988 and 1989, with presentations by Michael Oliva of the University of Wisconsin, Hota V. S. GangaRao of the West Virginia University, Ralph Mozingo of Pennsylvania State University, and David Pollard of the National Forest Products Associations (NFPA). Strategies have been set to present specifications for stressed slab decks, T-sections, and parallel-chord and boxed-girder structures.

The timber industry in the United States has been disjointed in the past, so the NFPA established a timber bridge task force that has started to coordinate national efforts in the timber bridge arena. This task force will help the AASHTO Timber Bridge Technical Committee staff write specifications and do public relations for timber bridges. The task force is made up of experts from industry, academia, and government.

The Forest Service published a book, *Timber Bridges—Design, Construction, Inspection, and Maintenance*. This excellent book will offer guidance to the practicing engineer for designing all types of timber bridges. It is deficient on design of stressed-timber bridges. Hota GangaRao of West Virginia University and the Forest Products Laboratory is working on specifications for stressed-timber bridges, slab decks, T-sections, and boxed girders to supplement the book.

The timber bridge technical committees are seeking greater involvement from the ASCE, TRB, AASHTO, and NFPA. ASCE has selected Paul Nicholas of Trus-Joist Corporation as chairman of the Timber Bridge Committee. Don Flemming of the Minnesota Department of Transportation has been doing an excellent job for the AASHTO Timber Bridge Technical Committee. The TRB timber bridge subcommittee met for the first time at the January 1988 TRB meeting in Washington, D.C. Papers were given by Mike Oliva, University of Wisconsin; Mike Ritter, U.S. Forest Service; and Ken Johnson, Wheeler Consolidated, Inc. Weller presented a paper at the January 1989 TRB meeting of the Structures Committee, which is concerned with current technologies in timber bridges. As mentioned, the NFPA Timber Bridge Task Force has recently become a driving force in timber bridges, and will be expected to provide continued leadership in the technology transfer arena. Tom Williamson of the American Institute of Timber Construction chairs that task force.

STRESSED-TIMBER BRIDGES

Description

Stressed-timber bridge superstructures consist of timber planks of varying lengths, longitudinally laminated on edge, butt jointed to allow for short members, and transversely stressed with high-tension stressing rods (150 ksi) to achieve an orthotropic slab, T-section, boxed-girder, or parallel-chord structure. These structures can be used for buildings as well as the bridges for which the concept was conceived.

History

Csagoly and Taylor of the Ontario, Canada, Ministry of Transportation and Communication (MTC), conceived the concept when trying to rehabilitate delaminated nail-laminated timber decks. The MTC researched the subject extensively. In 1984, the time was ripe to incorporate and expand the stressed-timber concept in the United States. The Forest Service performed additional research through the Forest Products Laboratory in Wisconsin and through the University of Wisconsin. The first bridge fabricating company to investigate and incorporate stressed-timber bridges in the United States was Wheeler Consolidated, Inc. Research in the United States has been done at Pennsylvania State University, West Virginia University, Colorado State University, and several other universities.

Concept and Existing Bridges

Stressed-wood bridges compare with the glue-laminated wood bridges that are fabricated into panels of 4-ft widths and jointed in the field with spreader beams or close-fitted dowels. Stressed-wood bridges can also be compared to reinforced, stressed-concrete structures. Total composite action is achieved through the stressing process, which is not easily achieved with glue-laminated wood panels. The composite action is achieved through the high friction forces created between the laminates under stress. Of the many stressed-timber bridges in the United States, configurations such as slab decks, parallel chord, T-sections, and boxed girders exist. Composite action has been achieved with all of these configurations.

As part of the national timber bridge conferences at Pennsylvania State and Colorado State Universities, Wheeler Consolidated, Inc., of St. Louis Park, Minnesota, constructed two stressed-timber slab decks. In addition, the state of West Virginia and West Virginia University have constructed, and are monitoring the performance of, a stressed T-section timber bridge in which they used hardwoods (red oak) for decking. West Virginia Department of Transportation has used GangaRao's designs for slab decks, T-sections, and boxed girders for over 30 bridges. Continued monitoring is being done. The Forest Service, as part of its service-wide technology transfer plan involving coordination with the U.S. Department of Agriculture, FHWA, NFPA, states, and universities, is continuing to collect data and evaluate the performance of many of the 100 timber (mostly stressed-timber) bridges that were part of the timber bridge initiative.

On the 16-in.-deep, 46-ft-span Pennsylvania State University bridge shown in Figure 1, the relaxation in the stressing rods stabilized within 2 months and the bridge is performing well. No cracks exist in the 2- to 3-in. asphalt wearing surface. Ralph Mozingo of Pennsylvania State University designed and constructed the prototype reinforced stressed structure that he feels can make the bridge less limber while decreasing the slab thickness, thus reducing costs. The Colorado State University bridge is a 40-ft span with 4- by 14-in. laminae.

Both for the Pennsylvania State and Colorado State bridges, Wheeler Consolidated, Inc., used 8-, 12-, 16-, and 20-ft-long planks. The design requires three of four laminae to be continuous at any point. Some bridge engineers question whether the bridge is stiff enough. The deflection on these two bridges is about 1 in. with one highway load truck (or 1 in 600 stiffness). These bridges are two laned so deflection is 1 in 360 if both lanes are loaded (the odds of which are very low).

The Forest Service is exploring the use of parallel-chord trusses to increase longitudinal stiffness. At the University of Wisconsin at Madison, Mike Oliva, in cooperation with the Forest Service's Forest Products Laboratory, has been testing many configurations of the stressing rods. The individual trusses are made of 4-in. by 6- to 12-in. chord members of the 12-, 16-, and 20-ft lengths, and the webs are 4- by 12-in. by 4-ft blocks on 8-ft centers. The chords are connected to the web with ⅜-in.-wide by 18-in.-long dowel spikes. (These mechan-

FIGURE 1 Trout Creek Bridge, College Township, Pennsylvania.

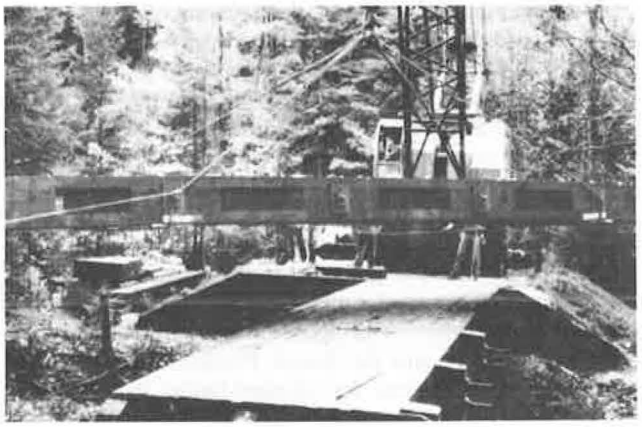

FIGURE 2 Parallel-chord stressed-timber bridge, Hiawatha National Forest, Michigan.

ical connections highly reduce the effectiveness of composite action achievable with stressed timber, and alternate solutions are being evaluated.)

The Forest Service constructed a 40-ft span demonstration bridge (shown under construction in Figure 2). Wheeler Consolidated, Inc., constructed the bridge superstructure in two panels and delivered them to the site for a simple field installation of 3 hr by novice Forest Service employees.

The Chequamegon National Forest also installed a two-span, 36-ft-long, continuous 8-in.-deep stressed slab deck on existing abutments and piers. The Forest Products Laboratory should be able to evaluate potential composite action. This continuous slab will help increase longitudinal stiffness while reducing deflection.

The Forest Service's northern region constructed a 30-ft span stressed microlam slab deck on the Idaho Panhandle National Forest. Trus-Joist has designed the stressed microlam superstructure similar to T-section concrete panels. These panels can be shop fabricated for simple field installation. Figure 3 shows the profile and cross-section views of the stressed

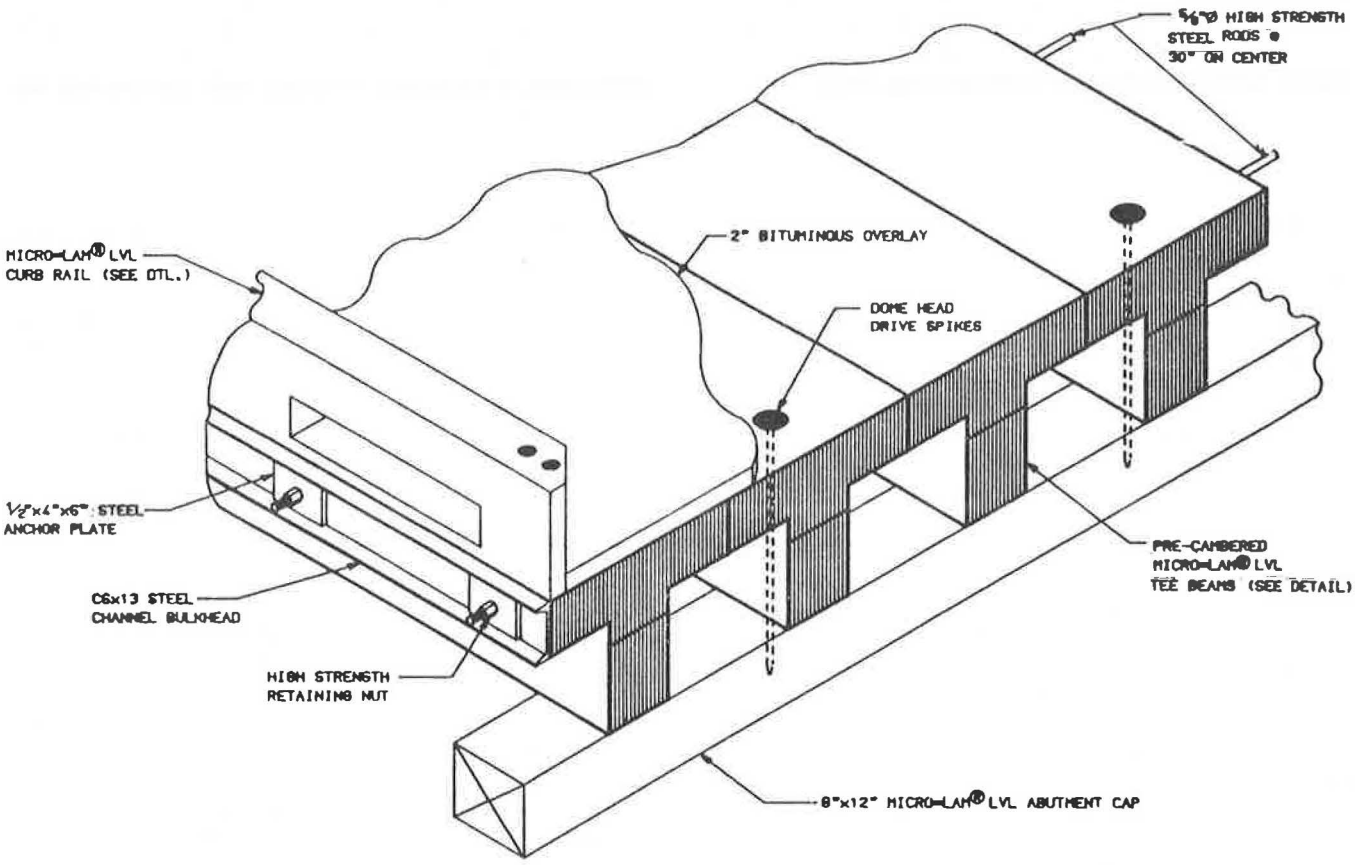

FIGURE 3 Pictorial view of bridge components.

microlaminated T-section design. A demonstration project of the stressed T-section laminated veneer lumber was constructed in Idaho as part of the Timber Bridge Conference in 1989, the bridge was 47 ft long with a 41-degree skew.

The state of West Virginia constructed a T-section stressed-timber bridge as part of the May 1988 Timber Bridge Conference. The red oak decking is stressed integrally with the laminated veneer lumber stringers that are 6 by 42 in. in cross section. The red oak deck laminates are 2 by 9 in. Load tests on the bridge indicate that this composite action is achieved in the field. Deflections from a recent load test show less than ½ in. with a 26-ton HS–20 load on the bridge. Longitudinal stiffness is about L/900 with an HS–20 load. A photograph of the 73-ft span bridge is shown in Figure 4. The results were predicted by the research at West Virginia University conducted by Hota V. S. GangaRao. Total composite action between the deck and stringers was achieved. The research was done in a 16-ft inverted W section using 2- by 6-in. red oak decking and 3- by 16-in. laminated veneer lumber stringers. Additional research was accomplished on a bulbed T-section (Figure 5). The bulbed T-section allows greater longitudinal stiffness and visual inspection of the underside of the superstructure.

FIGURE 4 Charleston, West Virginia, stressed T-section timber bridge, 73-ft span.

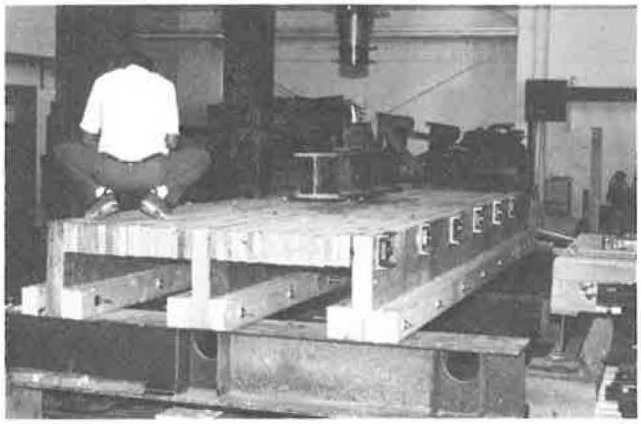

FIGURE 5 Stressed bulbed T-section model at West Virginia University, 16-ft span with 2- by 6-in. decking and 3- by 16-in. stringers.

West Virginia University has researched a stressed boxed girder that exhibits longitudinal stiffnesses three times those of the stressed T-section. (This result is easily predictable by calculating relative moments of inertia.) The laboratory study was performed on the section shown in Figure 6. West Virginia Department of Transportation has constructed six GangaRao-designed, stressed, boxed-girder bridges, and deflections as predicted in the laboratory have been observed. One of the first prototype stressed, boxed-girder systems was the Field Crest Bridge near Morgantown, West Virginia (see Figure 7). Figure 8 shows the two prefabricated boxed-girder panels of the Field Crest Bridge being placed on the abutments.

In 1989, Wheeler Consolidated, Inc., constructed a continuous 150-ft stressed deck over three spans and plans another bridge with a 200-ft continuous stressed deck over several spans in the Dakotas. The 150-ft deck was constructed on the road by the bridge site and launched over the abutments and piers (Figure 9). Composite action was accomplished with the steel girders. Drilled recessed holes in the deck were grouted to studs welded to the girders' wide flanges. The deck was then stressed further, ensuring the composite action. The deck of the continuous stressed deck picturing the grouted, drilled

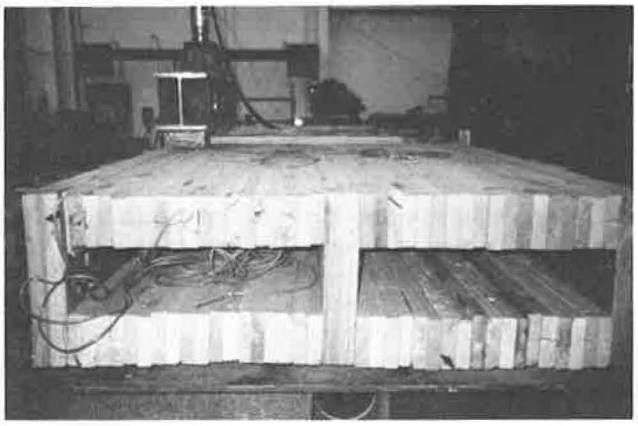

FIGURE 6 Stressed boxed-girder model at West Virginia University, 16-ft span with 2- by 6-in. top and bottom flanges with 3- by 16-in. webs, which are continuous through the top and bottom flanges.

FIGURE 7 Stressed boxed-girder timber of Field Crest Bridge, Morgantown, West Virginia.

FIGURE 8 Prefabricated, stressed, boxed-girder bridge panels being lifted onto abutments at Field Crest Bridge, Morgantown, West Virginia.

recessed holes for composite action on the steel stringers is shown in Figure 10.

In all, the United States now has more than 100 stressed-timber bridges, as compared to just 5 years ago when it had none.

CONCLUSION

In summary, timber bridges have been revived in the United States. Stressed timber can reduce the historical deficiencies of wood for building bridges. Stressed timber is less sensitive to moisture, decay, and joints. The composite action displayed by stressed timber tremendously increases load distribution and load-carrying capabilities of wood bridges.

Stressed timber plus other traditional designs will help fill the need for bridge replacements for the 240,000 deficient bridges in the United States and for other countries with similar bridge problems.

FIGURE 9 Continuous 150-ft stressed deck being readied to be launched or skidded onto the steel stringers.

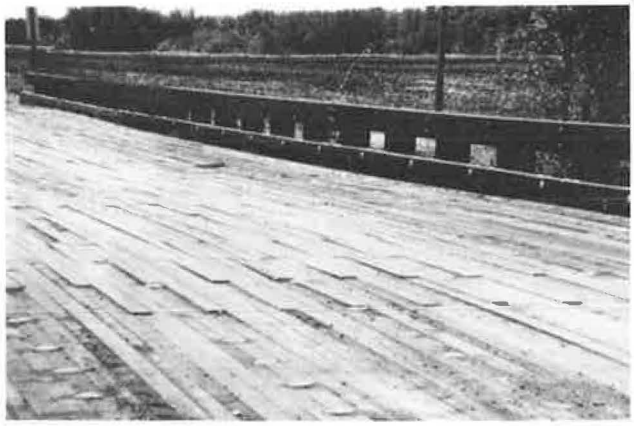

FIGURE 10 Deck with grouted, drilled recessed holes for composite action on a continuous 150-ft stressed-timber deck bridge.

System Innovation and Experimental Evaluation of Stressed-Timber Bridges

HOTA V. S. GANGARAO AND ISMAIL LATHEEF

Stressed decks are constructed by squeezing lumber laminations using galvanized steel bars with ultimate strengths of 155 ksi. Bridges of longer span lengths using stressed-deck concepts are feasible with two different methods: (a) increasing the number of layers of lumber laminations of the stressed deck; and (b) adding stringers and diaphragms to the stressed deck. The stressed-bridge systems developed and presented include (a) slab or solid stressed deck, (b) tee, (c) bulb-tee, and (d) box. Only tee, bulb-tee, and box systems were tested. The tee, bulb-tee, and box with laminated veneer lumber (LVL), glulam, and fiber-reinforced plastic stringers are tested for bending under three different load conditions and their responses are evaluated. The significant performance characteristics (such as lamination separation caused by transverse bending, vertical slip caused by direct vertical load, lamination rotational slip at butt joints caused by longitudinal bending, and the minimum transverse prestress levels in laminations) are checked. The tested systems (tee, bulb-tee, and box) are compared for their stiffness. The systems' composite action, diaphragm effects, and load sharing are evaluated. In addition, ultimate load characteristics are studied by taking the specimens to failure. GangaRao's analytical model with orthotropic material properties is verified with the experimental results.

In stressed decks, lumber laminations with a nominal thickness of 2 in. are squeezed by high-strength steel bars to make the resulting structure act like a plate. Frictional forces are created on the faces of lumber laminations caused by normal compression stresses created by prestressing forces. These frictional forces are responsible for preventing the lumber laminations from slipping vertically off their contact faces. When loads are applied perpendicular to the plane of the deck, laminations may separate from the contact faces at bottom fibers. If an adequate amount of compressive stresses are maintained on the contact faces of laminations, this problem can be prevented. The research conducted at the University of Wisconsin and at West Virginia University (WVU) indicates that a prestress level of 50 psi is adequate to prevent vertical slip and separation of laminations (*1–3*). Butt joints of lumber laminations can be provided in stressed decks because the frictional stresses on contact faces can transmit bending stresses from one laminate to another even at butt joint locations. A solid stressed deck can be modeled as an orthotropic plate with different properties in the longitudinal and transverse directions.

Stressed-slab decks are safe and economical for bridges of less than 30-ft span. (This result is based on the construction projects of 33 timber bridges in West Virginia during the years 1989 and 1990.) However, use of stressed-slab decks may be uneconomical for bridges with spans greater than 35 ft. Bridges of higher span lengths using stressed decks are possible with two different methods: (a) increasing the numbers of layers of lumber laminations of the stressed deck, and (b) adding stringers to the stressed deck. In addition to stringers, diaphragms could be used in between stringers for increasing the lateral stability of stringers, improving the load distribution between stringers, and increasing the bending and shear resistance of the overall systems. A variety of manufactured materials such as LVL, glulam, parallam, and fiber-reinforced plastic (FRP), including steel and aluminum, can be used for stringers and diaphragms. Therefore, stressed systems for timber bridges for longer spans (more than 35 ft) are possible by modifying the basic stressed-deck component. Also, other alternate systems can be configured by the geometries and materials of the components to choose the system to best suit the design requirements and site conditions.

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STRESSED-TIMBER BRIDGE SYSTEMS

In a manner similar to concrete and steel bridges, stressed-timber bridge systems can be developed into structurally efficient and cost-competitive superstructures for primary and secondary highway applications. In the natural order of development for stressed-timber bridge systems, the solid deck system was developed first; then the design was modified to include stringers and diaphragms. Each stressed-timber bridge system is described. The dimensions that are shown in Figures 1–4 are the actual dimensions of the tested specimens.

Solid Stressed Deck

A solid stressed deck can be composed of single- or multiple-layer laminations through its depth. Figures 5 and 6 show the cross-sectional details of single- and multilayer stressed decks, respectively. In a multilayer solid stressed deck, different sizes of laminae are placed in such a way that an interlocking mechanism is created between them.

Tee Systems

The tee systems consist of stringers and flange components compressed together by the stressed rods as shown in Figure 1. The compressive force creates a composite action between the stringers and the flange by developing frictional forces between the contact faces of the components. Diaphragms are added (Figure 2) in between stringers for the tee systems

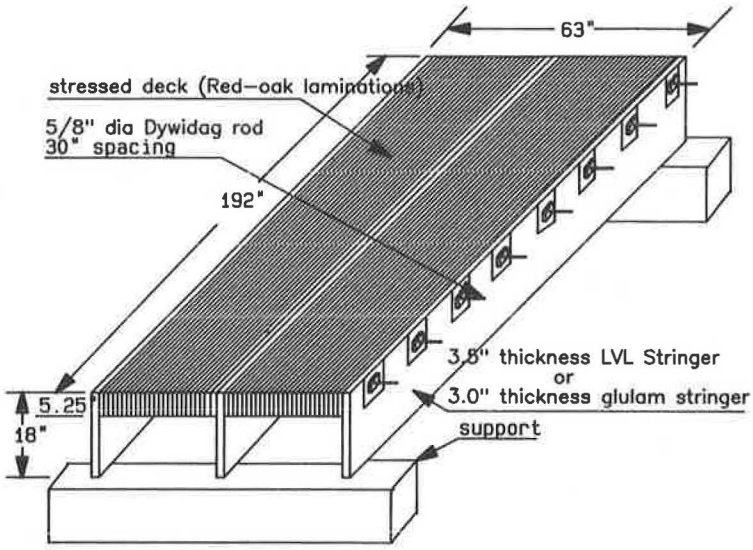

FIGURE 1 Tee system specimen.

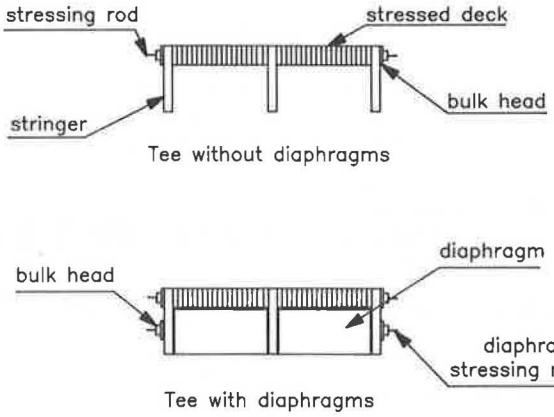

FIGURE 2 Tee system with and without diaphragms.

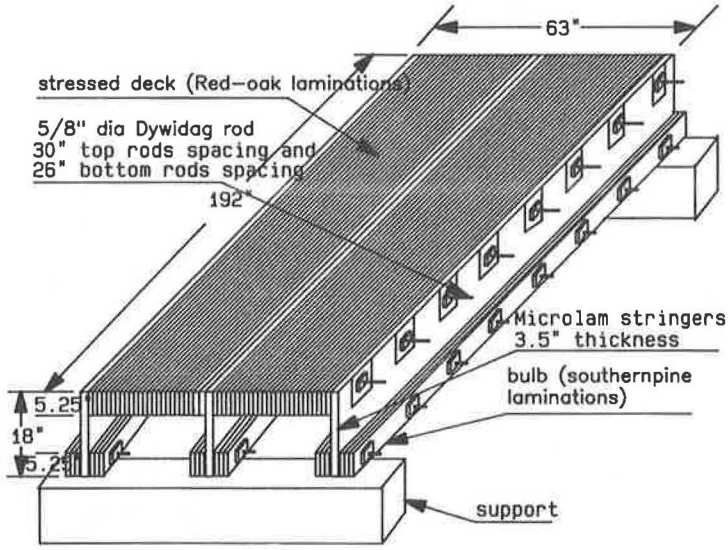

FIGURE 3 Bulb-tee system specimen.

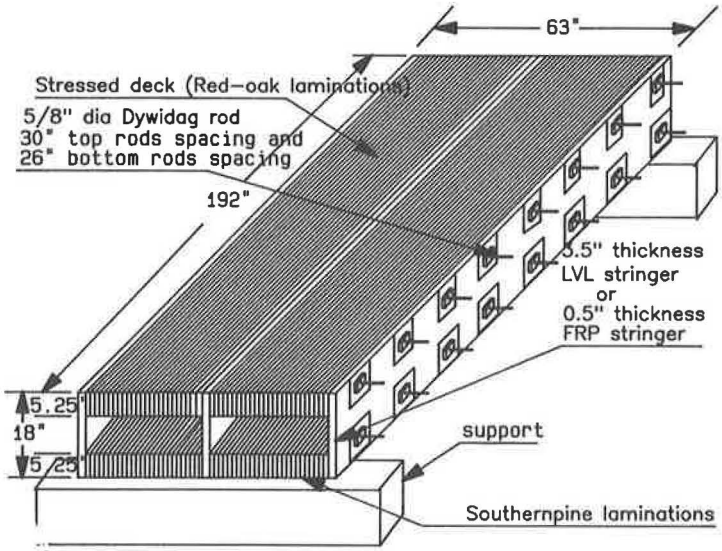

FIGURE 4 Box system specimen.

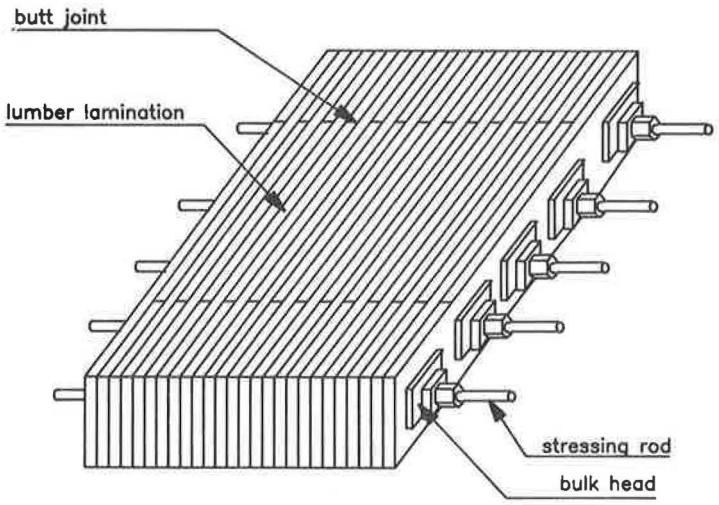

FIGURE 5 Solid stressed deck (single layer).

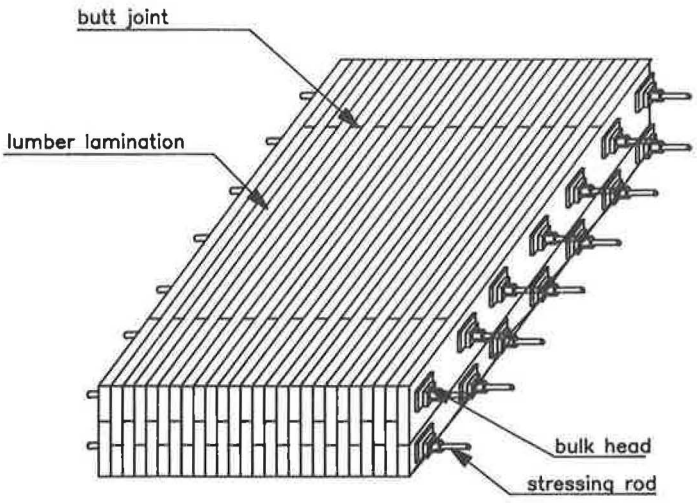

FIGURE 6 Solid stressed deck (multilayer).

to prevent lateral-torsional buckling of the stringers in case of large spans and for better transverse load distribution and improved stiffness.

Bulb-Tee Systems

The bulb-tee system is similar to a tee system, except that timber planks are attached at the bottom of each stringer to create a bulb. Figure 3 shows the details of this design. Threaded rods are used to hold the bulb laminations. These rods are prestressed using a hydraulic jack. The bulb-tee stringers can also be constructed with diaphragms to prevent lateral buckling, as described earlier in this section.

Box Systems

Figure 4 shows the design of a cellular box system. This system is stiffer than girder sections described previously and requires shallower stringer depths than a tee system. However, a major disadvantage of a cellular box design is the difficulty of visual inspection. In addition, the closed environment of a box may trap moisture and lead to moisture condensation on the interior walls of the cells. In the case of large stringer depths, as with the tee and bulb-tee systems, diaphragms could be added to prevent buckling of stringers.

Alternate Systems

An optional design may require considering alternate timber components that best suit the site requirements. The alternate systems could be developed by simply altering the stringer materials and changing the diaphragm options. The possible stringer materials are steel, aluminum, LVL, glulam, parallam, and FRP. The diaphragms can be connected to the stringers with stressing rods. Holes have to be drilled all the way through the entire length of the diaphragms and the width of the stringers in order to run the stressing rods from one end to the other. The location and number of diaphragms are primarily controlled by design load requirements.

EXPERIMENTAL AND THEORETICAL ANALYSIS

Full-scale stressed-timber systems were tested in the WVU Major Units Laboratory. The test specimens include tee, bulb-tee, and box specimens with LVL, glulam, and FRP stringers, with options of diaphragms, and also individual LVL and glulam stringers. Theoretical analysis of stressed systems is mainly focused on the load-sharing capacity, deflections, and strains of the stringers. In the analysis, GangaRao's orthotropic plate equations are used (*4*). GangaRao's equations were basically derived from the analysis of bridge decks idealized as ribbed or grid-plate systems. Because the stressed timber systems are similar to ribbed systems, the formula for ribbed systems can be extended to the analysis of stressed-timber systems.

OBJECTIVES

The objectives of the experimental evaluation program were

1. To test tee, bulb-tee, and box systems (Figures 1–4) under different load conditions with the objective of establishing degree of composite action, influence of diaphragms, characteristics of load distribution, induced strains and stresses, and load deflection responses.
2. To check the stressed deck elements (hardwood planks) for possible adverse rotation of butt joints, vertical slip (incomplete shear transfer), and separation on the tension side of the deck under applied loads.
3. To test in bending the bridge components (LVL and glulam stringers) and the fully assembled systems (tee, bulb-tee, and box) in order to assess the failure mode and transverse load distribution characteristics of the systems.
4. To evaluate the magnitude of experimental responses, such as deflections and strains, of the three stressed systems under applied loads.
5. To obtain analytical results, such as transverse load distributions, stresses, and deflections, using GangaRao's orthotropic plate solutions, and correlate them with experimental results.

EXPERIMENTAL PROGRAMS

The test specimens for tee, bulb-tee, and box systems are shown in Figures 1, 3, and 4, respectively. These specimens consist of red oak top flanges and Southern pine bottom flanges (when required). The grade of the lumber used was No. 3. The systems were constructed with stringer components consisting of three distinct materials: LVL, glulam, and FRP. The FRP stringers had 50 percent of E-Glass fiber and 50 percent of vinylester matrix. The systems that were tested are presented in Table 1. Dywidag rods of $\frac{5}{8}$ in. diameter were used to prestress the system, and a 100-psi prestress level in timber was maintained in the test specimens. The diaphragms are attached to the stringers by the compressive action of the prestressing rods. Three types of static load conditions were applied, referred to as load configurations *A*, *B*, and *C* as shown in Figures 7–9. The specimens were subjected to load conditions *A*, *B*, and *C*, and their load-deflection and load-strain responses were recorded. Strain gages and dial gages were used to measure strains and deflections. Loads were applied gradually and the loading range varied from 0 to 60 kips. The following systems were tested to failure: bulb-tee with diaphragms, glulam tee without diaphragms, and FRP box with diaphragms. For these tests, the load level was continuously increased even after the occurrence of the first failure of one or more of the stringers. Descriptions of the tests are presented in Table 1. In addition, the individual LVL and glulam stringers were tested in bending in accordance with the ASTM D198–84E standard. The specimens were simply supported and subjected to a concentrated load at the center. In order to prevent torsional buckling, lateral supports were

TABLE 1 LIST OF SPECIMENS TESTED UNDER DIFFERENT LOAD CONDITIONS

System	Tested Loading Conditions	Remarks
Tee with LVL Stringers with Partial dia-phragms	A	-
Bulb-Tee with LVL Stringers w/o Diaphragms	A	-
Bulb-Tee with LVL Stringers with Diaphragms	A	The specimen was tested to failure.
Box with LVL Stringers w/o Diaphragms	A,B,C	-
Box with LVL Stringers with Diaphragms	A	-
Tee with Glulam Stringers w/o Diaphragms	B	-
Box with FRP Stringers w/o Diaphragms	A,B	The specimen was tested to failure.
Box with FRP Stringers with Diaphragms	A	The specimen was tested to failure.

(-) - Specimen was not tested to failure.

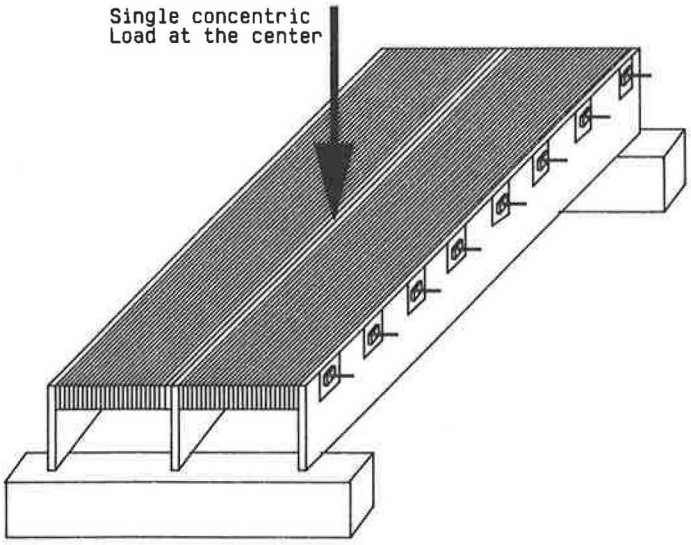

FIGURE 7 Load condition A.

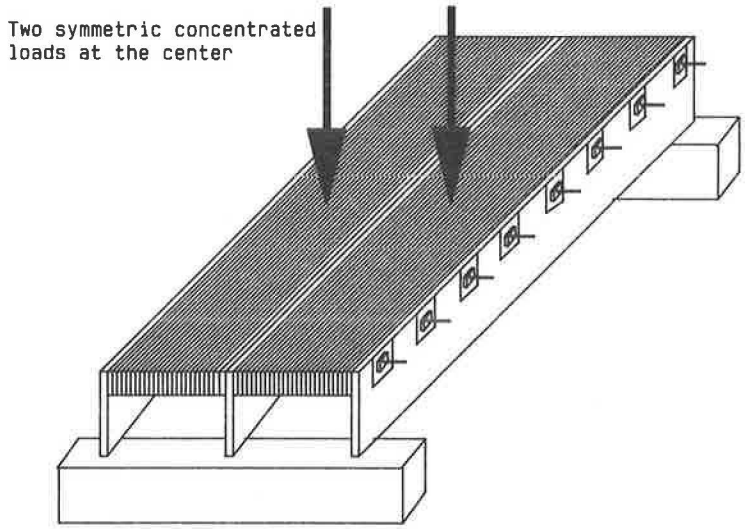

FIGURE 8 Load condition B.

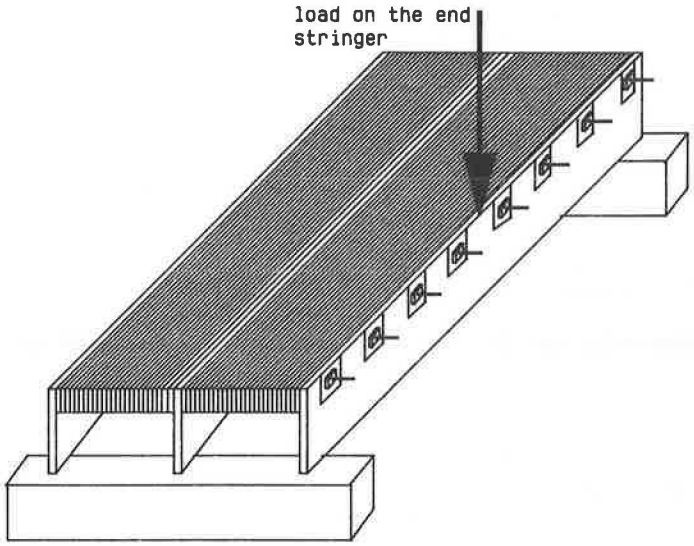

FIGURE 9 Load condition C.

provided for the specimens. The ultimate bending stresses were established by loading the stringer specimens to failure.

DISCUSSION OF RESULTS

Composite Action

Composite action of the stressed-timber systems considered in this report was verified by establishing the location of the neutral axis from top and bottom fiber readings in the longitudinal direction of stringers. Table 2 presents the shift of the neutral axis of the interior stringer from the geometric centroidal axis. For the tee and box systems, neutral axes were found to be 2.41 and 0.43 in., respectively, away from the geometric centroidal axis of their corresponding stringers. Computed shifts of the neutral axes of those sections on the basis of the classical static approach were close to the experimental values presented in Table 2. On the basis of the results, it can be concluded that nearly full composite action is developed for loads within the linear range. However, the composite action does not appear to be 100 percent for loads approaching failure. For example, when testing the tee section, the central stringer failed first; however, the adjacent deck laminations did not show any distress. Also, when the FRP box section was loaded to failure, the system underwent large deflections (over 6 in. at midspan) without any failure of the deck laminations occurring.

Load Deflection Characteristics of Stressed Systems

The load deflection curves demonstrate linear and nonlinear responses, which depend on the magnitudes and locations of the applied loads. For example, the tee with glulam stringers as tested to failure exhibited nonlinear behavior for load con-

TABLE 2 SHIFTING OF NEUTRAL AXIS BECAUSE OF COMPOSITE ACTION

Type of System and Stringer Location	Strain on top of the stringer $(in/in)*10^{-6}$	Strain on bottom of the stringer $(in/in)*10^{-6}$	Shifting of the neutral axis from the geometric center of the stringer	
			Experiment	Analytical Approach
Tee with LVL Stringers (without diaphragms)-Interior Stringer	-1270	2200	2.41"	1.61"
Box with LVL Stringers (without diaphragms)-interior stringer	-465	510	0.43"	0.34"

dition B exceeding 30 kips (Figure 10). The bulb-tee with LVL stringers and diaphragms showed nonlinear behavior when loaded beyond 20 kips under load condition B (Figure 11). However, for load condition A, nonlinear behavior was not observed even at a load level of 30 kips (Figures 12–14). These observations imply that the lower transverse stiffness of the flanges causes them to undergo large local deflections when the load is applied directly on them. In contrast to the timber box section, the FRP box system showed nonlinear behavior both for load conditions A and B, which can be attributed to the low flexural stiffnesses of the FRP stringers (Figures 15 and 16); i.e., geometric nonlinearity of the members is probably the major cause of the nonlinear response of the system, which is magnified by the presence of butt joints and the opening of laminations for load levels above 20 kips. From the test results, it is evident that every stressed timber system will exhibit a degree of nonlinear response beyond a certain load range. In order to satisfy the serviceability limit states, design loads have to be within this load range.

Influence of Diaphragms

In Table 3, the addition of diaphragms to the bulb-tee system (with LVL stringers) reduced the maximum deflection from 0.895 to 0.593 in. for a 30-kip load, which represents an increase of 50 percent in the stiffness of the system. The box system (with LVL stringers) did not show any increase in stiffness

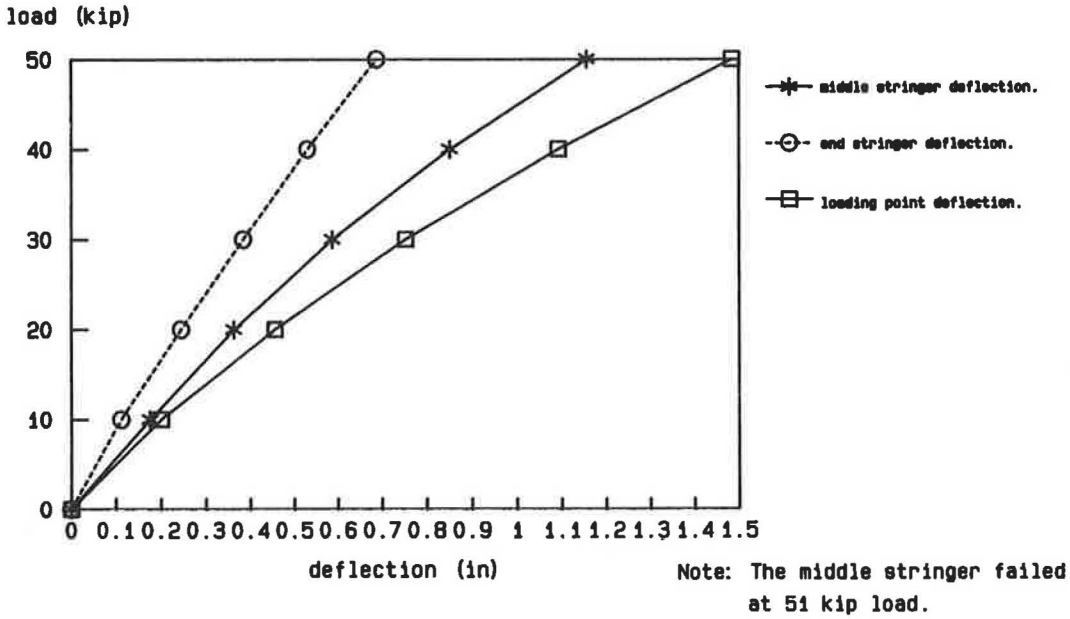

FIGURE 10 Load-deflection curves for the tee with glulam stringers (without diaphragms) for the Load Condition B.

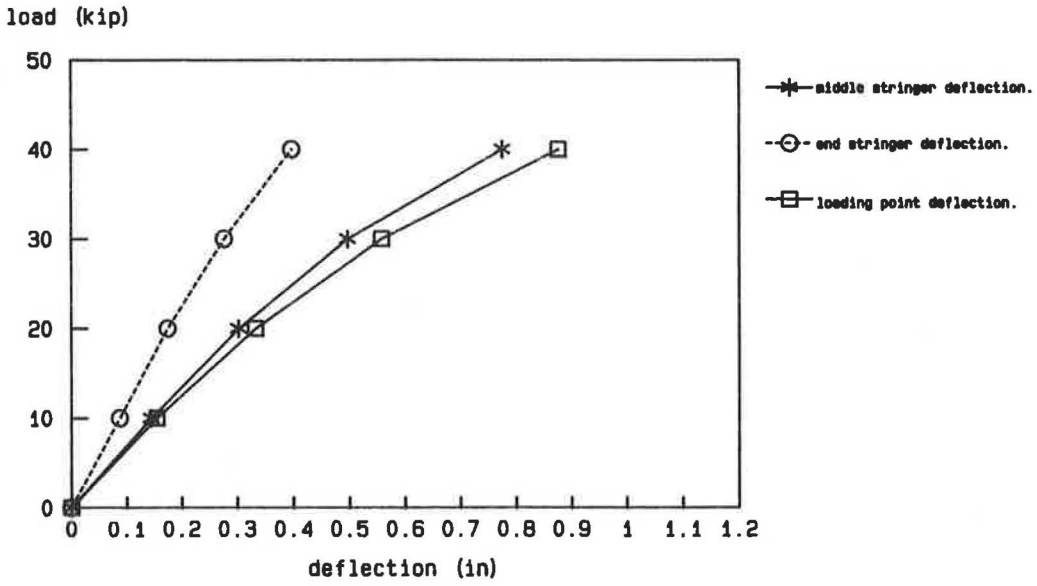

FIGURE 11 Load-deflection curves for the bulb-tee with LVL stringers (with diaphragms) after the failure of the middle stringer for Load Condition B.

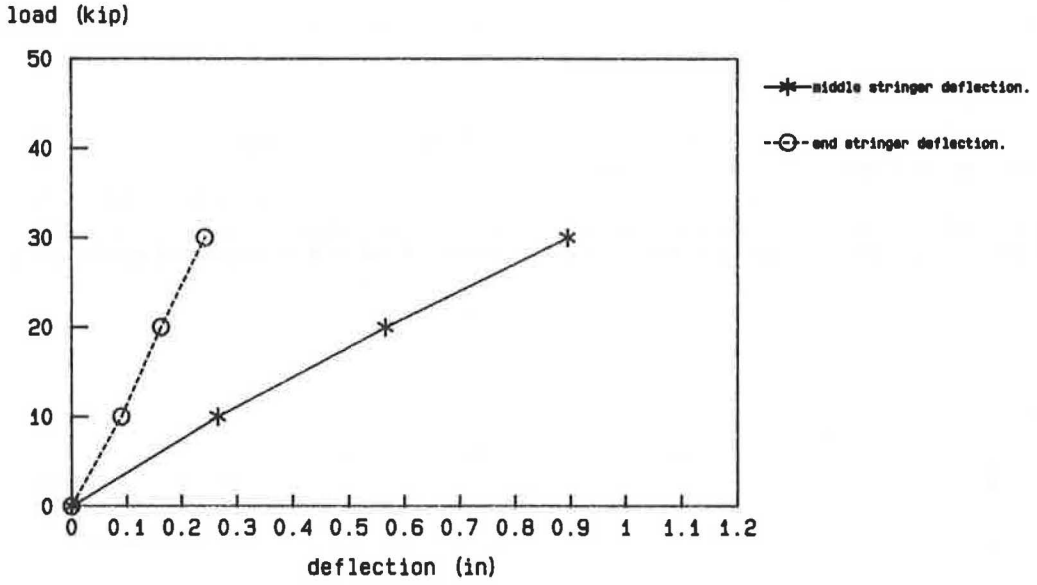

FIGURE 12 Load-deflection curves for the bulb-tee with LVL stringers (without diaphragms) for Load Condition A.

when the diaphragms were added, which can be attributed to the fact that the deck itself acts as a diaphragm for shallow stringer depths. The addition of diaphragms to box system (with FRP stringers) increased the deflection from 0.780 to 0.974 in., which is attributed to readjustment of the butt joints of lumber laminations and FRP stringers when diaphragms were present.

Failure Modes of Systems and Individual Components

The bulb-tee with LVL stringers and diaphragms was tested to failure by applying a concentrated load (load condition A) to the middle stringer. A tensile bending failure to the middle stringer was observed at a load of 60 kips (corresponding to 4,350-psi flexural stress). The tee with glulam stringers was tested to failure under load condition B. The middle stringer failed in bending at a load level of 51 kips (4,800-psi tensile bending stress). The failure was sudden and was originated at a knot located at the outer bottom fiber. The box with FRP stringers and timber diaphragms was tested to failure under load condition B. The specimen did not show any failure or vertical slippage of the elements up to a load level of 50 kips. For a sustained load of 58 kips, the specimen failed because of buckling failure of the compression flanges of the FRP stringers. In the tested systems, no shear failure of the string-

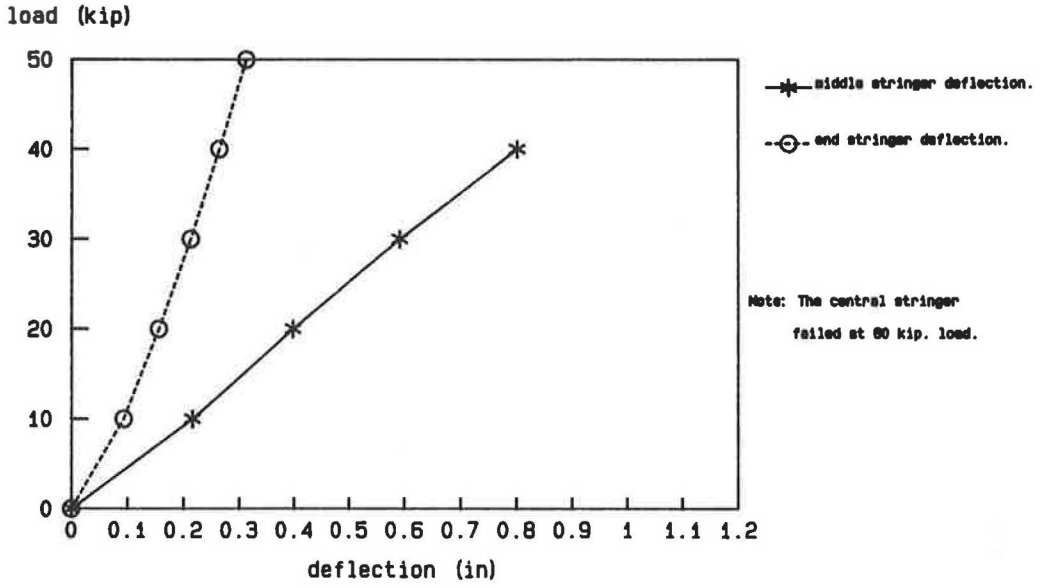

FIGURE 13 Load-deflection curves for the bulb-tee with LVL stringers (with diaphragms) for Load Condition A.

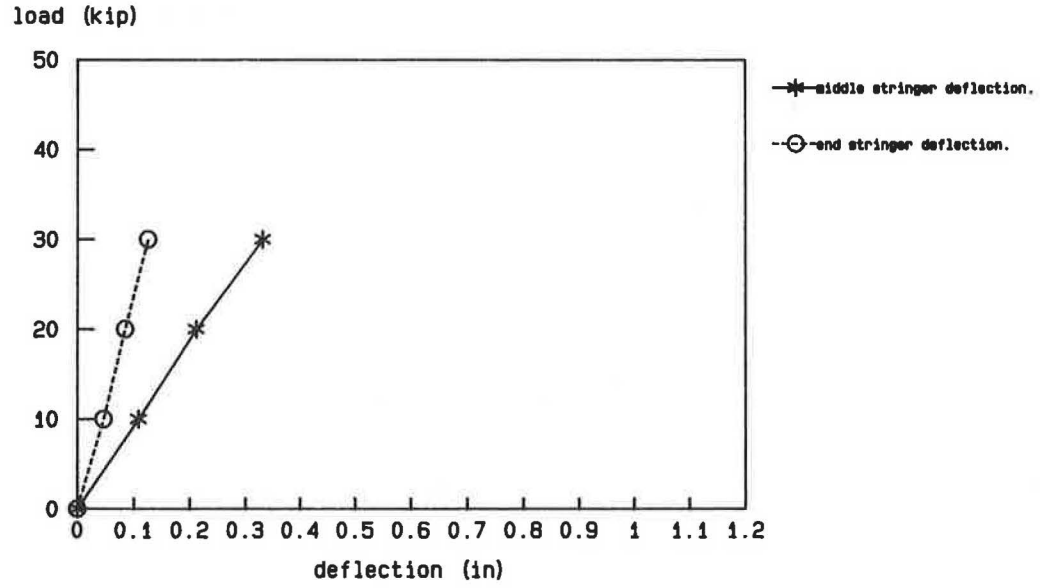

FIGURE 14 Load-deflection curves for the box with LVL stringers (without diaphragms) for Load Condition A.

ers and no failure or vertical slippage of the laminations was observed. Testing of individual LVL and glulam stringers showed bending failure at the bottom-most fiber at midspan. The LVL and glulam stringers failed at 25.00 kips (4,350-psi tensile bending stress) and 25.60 kips (4,800-psi tensile bending stress), respectively.

Assessment of Load Sharing of Stringers

The percentage of load sharing of the stringers was evaluated by comparing the test results of the ultimate loads of the stressed systems and the individual stringers. Table 4 indicates that the load shared by the stringers without diaphragms is 50 percent of the total load carried by the system, and when diaphragms are included, the percentage is 58 percent.

Stiffness Comparison of Stressed System

The maximum deflections of the system considered in this study, for a 30-kip load and load condition A, are presented in Table 5. The relative stiffnesses for the bulb-tee systems with and without diaphragms, the box systems, and all of the

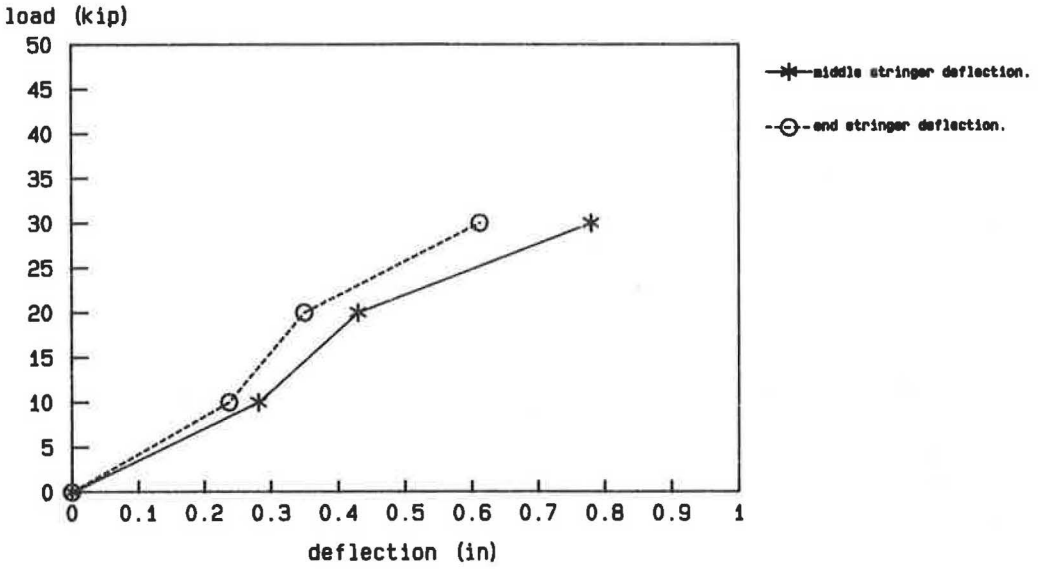

FIGURE 15 Load-deflection curves for the box with FRP stringers (without diaphragms) for Load Condition A.

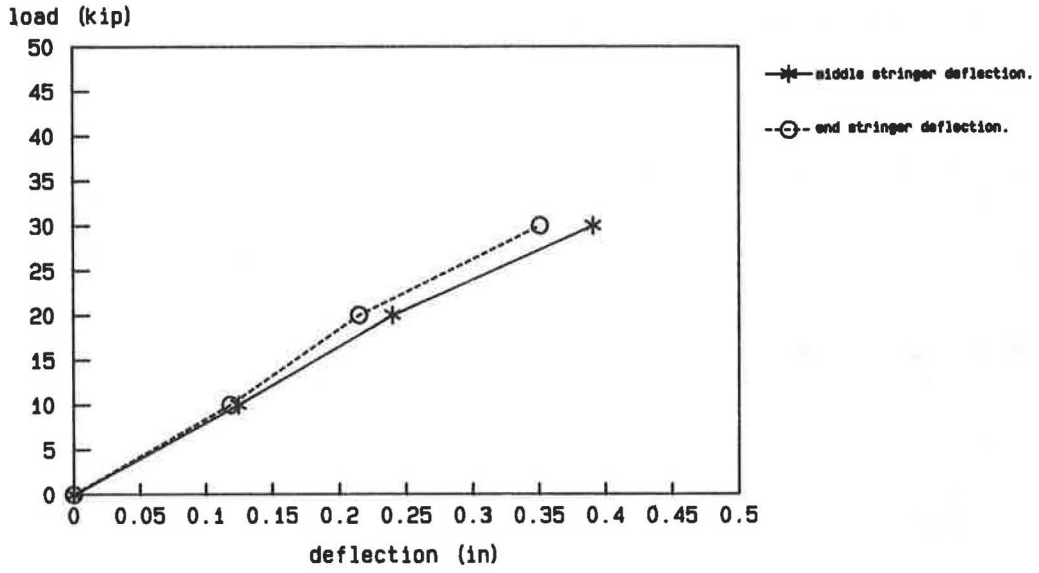

FIGURE 16 Load-deflection curves for the box with FRP stringers (without diaphragms) for Load Condition B.

TABLE 3 STIFFNESS COMPARISON WITH ADDITION OF DIAPHRAGMS

System	Deflection Without Diaphragms (in)	Deflection With Diaphragms (in)	% Increase in Stiffness
Bulb-Tee with LVL Stringers	.895	.593	50%
Box with LVL Stringer	.332	.350	00%
Box with FRP Stringer	.780	.974	00%

TABLE 4 PERCENTAGE LOAD SHARING OF MIDDLE EXTERIOR STRINGER

System	Type of Stringer	Ultimate Load of the Stringer	Load Condition of the System	Ultimate Load of the System	% Load Carried by the Middle Stringer	% Load carried by the Exterior Stringers
Bulb-tee with LVL Stringer with Diaphragms	LVL	25 kip	A	60 kip	42%	58%
Tee with Glulam Stringer without diaphragms	glulam	25.6 kip	B	51 kip	50%	50%

TABLE 5 MAXIMUM DEFLECTION OF THE STRESSED SYSTEMS FOR A 30-KIP LOAD

System	Deflection (inch)		
	Load Condition A	Load Condition B	Load Condition C
Tee with LVL Stringers with partial diaphragms	.816 (A_2)	-	-
Bulb tee with LVL-Stringers without diaphragms	.895 (A_2)	-	-
Bulb Tee with LVL-Stringers with Diaphragms	.593 (A_2)	-	-
Box with LVL-Stringers without diaphragms	.332 (A_2)	.249 (b_2)	.501 (b_2)
Box with LVL-Stringers with Diaphragms	.350 (A_2)	-	-
Tee with Glulam Stringers without diaphragms	.753 (Interpolated Value) (A_2)	.737 (C_2)	-
Box with FRP Stringers without diaphragms	.780 (A_2)	-	-
Box with FRP Stringers with Diaphragms	.974 (A_2)	-	-

A_2 - Center point at the top of the middle stringer.

b_2 - Center point at the top of the exterior stringer.

C_2 - Bottom of the loading point of the load condition B.

(-) - Test was not conducted

other systems considered in this study are consistent with expected results. The inclusion of diaphragms in the bulb-tee system reduced the maximum deflection from 0.895 to 0.593 in. The maximum deflections of the box system are 0.243 and 0.486 less than those of the tee and bulb-tee systems, respectively. The deflection values for the box with LVL stringers and no diaphragms (0.332 in.) and the box with LVL stringers and diaphragms (0.335 in.) are nearly identical, which implies that the diaphragm action in the box system with shallow stringer depths is negligible. The deflection of the box with FRP stringers with diaphragms (0.974 in.) is more than the maximum deflection of the same box without diaphragms (0.780 in.), which was attributed to readjustment of butt joints of the laminations and the FRP stringers when diaphragms were present.

Discussion of Special Topics

Stressed-timber systems have to be stressed at least three times to stabilize prestressing forces acting on timber and to minimize the time-dependent prestress losses. Research in Ontario (*5,6*) showed that when a constant transverse compressive stress is applied to a mechanically laminated timber deck system, it slowly deforms with time. If the deck system is stressed only once, a prestress loss of 80 percent or more would take place in a relatively short period of time before stabilization of prestressing forces on timber lamination. Subsequent stressing of the deck two or more times can reduce the long-term losses to a minimum, and a compressive prestress level of 50 lb/in^2 can be maintained. However, additional investigations have to be made regarding the effects of cyclic or fatigue loads on potential loss of prestress. The stressed systems that were tested in this study consisted of red oak (hardwood) and southern pine (softwood) decks. However, the findings provide a basis for the design of stressed systems with other hardwood and softwood combinations.

Theoretical Analysis of Stressed Systems

Table 6 presents a comparison between theoretical and experimental results. The differences between the theoretical and experimental results may be the result of using approximate values for the elastic constants in the computations. All test specimens had only three stringers (one interior and two exterior stringers). GangaRao's formulas more precisely represent systems with more than one interior stringer. Thus, the discrepancies in the results can also be partially attributed to the use of the equations for systems with just one interior stringer.

CONCLUSIONS

The following conclusions were drawn on the basis of limited data. More data are needed to substantiate the results.

TABLE 6 COMPARISON OF THEORETICAL AND EXPERIMENTAL RESULTS BETWEEN A TEE AND A BOX SYSTEM

Analysis Type	Deflection		Strain			
	Exterior Stringer	Interior Stringer	Top of ext. stringer	Bot. of ext. stringer	Top of int. stringer	Bot. of int. stringer
Tee System						
Orthotropic Plate Theory-WVU	0.233	0.691	503	775	1271	2518
WVU Method	0.174	0.485	455	703	1084	2147
Experimental Results	0.185	0.816	*	*	1348	2670
Box System						
Orthotropic Plate Theory-WVU	0.088	0.270	235	247	716	765
WVU Method	0.097	0.270	314	329	854	913
Experimental Results	0.123	0.330	237	165	465	570

* - Not measured

1. The flange component of the bulb-tee and box systems showed nearly full composite action with the stringers. However, there was no full composite action at load levels closer to failure.

2. Even under a 40-psi precompression stress, the lumber laminations of the flange components did not exhibit any vertical slip. Also, no opening or separation of the laminations in the transverse direction was observed. The butt joints showed no rotational slip, and no shear failure was observed in the flange elements or the individual stringers.

3. Depending on the load location, the stressed systems showed both linear and nonlinear load deflection responses. The nonlinear behavior was more pronounced at the locations of the concentrated loads and at the center of the interior stringer at very high loads, i.e., close to the failure loads. However, when the load was applied directly on the stringers, a linear behavior was observed. Also, the stressed systems demonstrated linear and nonlinear behavior depending on the overall stiffness of the systems.

4. The addition of diaphragms proved to be effective in improving the response of tee systems.

5. The percentage of load shared by the stringers of the tee system is approximately 50 percent, which increases to 58 percent when diaphragms are added. These percentages of load sharing could be higher for actual systems, which usually have more than three stringers.

6. When a system is loaded for the first time, it does not completely recover its deflection after removing the load. However, no such permanent deflection is noted during the second and third cycle of loading. This may be because of readjustment of the butt joints in the systems.

7. As expected, box systems showed higher stiffness value and lower stresses than tee and bulb-tee systems for a 30-kip load corresponding to load condition A.

8. The theoretical analysis based on GangaRao's plate solutions (*2*) demonstrated a good overall correlation with experimental results for the deflections and strains for the exterior and interior stringers of tee and box systems under load condition A.

ACKNOWLEDGMENTS

This research was cosponsored by the USDA–FPL, West Virginia Department of Highways, and USDA–FS. Materials were provided by the Burke, Parsons, Bowlby, Inc., and Dywidag Corp.

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Engineered Timber Systems for Short-Span Bridges

MERVIN O. ERIKSSON

Statistics reveal a large number of short-span bridges in this country, many of which are, or soon will be, in need of replacement. Timber has traditionally been used as a low-cost building material for short-span bridges on low-volume roads. With the development of superhighways during the past 3 or 4 decades, timber use in bridge construction has declined. Timber has good strength characteristics as well as being resistant to impact loading and damage from road salts. Proper preservative treatment can make wood a durable, long-term building material. Development of engineered timber products and systems has made modern timber bridges practical, reliable, and often the lowest-cost alternative. Products such as glued laminated beams and panels, laminated veneer lumber, and stressed deck systems have increased durability and feasible span lengths. Experience on USDA Forest Service roads has shown that the costs of timber bridges are consistently competitive for shorter spans, up to 30 ft, particularly on low-volume roads. Timber should be considered as a viable building material for many of the nation's deteriorating bridges.

Current figures, based on FHWA's January 1989 Coding Guide for the National Inventory of Bridges, show 225,462 substandard bridges in this country. These bridges are deficient for either structural or functional reasons; 140,552 of these bridges are under city, county, or township jurisdiction. Approximately 100,000 of these bridges are short-span bridges on low-volume roads (*1*). Adding federally owned bridges would push the low-volume-road short-span bridge total to more than 120,000 bridges. Existing bridge systems must be improved and new economical systems must be developed to meet the growing demand to replace these bridges. The timber bridge is a viable replacement option in many cases.

Timber is particularly suited to short-span bridges, especially spans shorter than 40 ft, and its low weight and simple fabrication make timber an excellent building material for rural, isolated locations. Timber's low modulus of elasticity (in relation to concrete and steel) and the difficulty of attaching a guardrail system often preclude its use on longer-span, high-volume, high-speed locations.

Wood is an adaptable, dependable construction material, which, because of its structural characteristics and widespread availability, has been used in construction for thousands of years. From early stone age time, when man crossed ravines and rivers on fallen logs, until the end of the 19th century, wood was the predominant bridge building material.

Properly designed, constructed, and maintained timber bridges can generally carry heavy loads without material fatigue, can resist the deteriorating action of deicing chemicals, can be constructed by unskilled labor, and will last for long periods of time. The covered bridges built in the 1800s are a part of the nation's heritage. Great timber truss bridges beyond 400 ft in span length and the magnificent railroad trestles of the Rocky Mountains testify to the skill of our early timber bridge designers and builders. There are approximately 1,000 mi of such railroad trestles still in use today in the United States.

Timber was one of man's first building materials. However, as modern 20th-century technology spurred development of new building products such as high-strength steel, aluminum, prestressed concrete, fiberglass, etc., the use of timber dropped. Timber is still the predominant building material in residential construction, but for the most part, the use of timber in highway bridge construction has dropped from its peak in the 1940s, to almost nothing in 1985. This drop in timber bridge design and construction has resulted in a shortage of knowledgeable, experienced timber bridge designers just when the United States is experiencing a crisis in the needed replacement of more than 120,000 short-span bridges. Timber should be considered a viable construction material for many of these local short-span bridges. Bridge designers should be informed of the viability of timber as bridge construction material and educated in proper timber design procedures. This is one of the major emphasis areas of the timber bridge initiative being administered by the USDA Forest Service (*2*).

Over 11,000 road bridges are maintained by the Forest Service, of which about 1,600 are located in the Northern Region (Montana and Idaho). Approximately 55 percent of the total has been constructed with timber superstructures (*3*). In the Northern Region, about 1,200 of the 1,600 bridges have been constructed using some type of timber bridge superstructure system. Most of these bridges also have treated timber substructures. The Forest Service designs and constructs 10 to 25 new bridges per year, about two-thirds of which use timber for either the superstructure or substructure.

CHARACTERISTICS OF WOOD

The fibrous nature of wood strongly influences how it is used. Specifically, wood is composed mostly of hollow, elongated, spindle-shaped cells that are arranged parallel to each other along the trunk of a tree. When lumber, or timber, is cut from the tree, the characteristics of these fibrous cells and their arrangement affect such properties as strength and shrinkage, as well as grain pattern of the wood. These cellulose cells vary in length from 1 mm for hardwoods to 8 mm for softwoods and comprise about 60 percent of the wood by volume. The thickness of the cell walls is the principal strength-determining characteristic of the wood. These hollow cells are cemented together by a water-soluble material called lignin.

USDA Forest Service, P.O. Box 7669, Missoula, Mont. 59807.

The water solubility of lignin causes reduced strength properties for wood with higher moisture content.

Wood may be described as an orthotropic material; that is, it has unique and independent strength properties in the direction of three mutually perpendicular axes—longitudinal, radial, and tangential. The longitudinal axis is parallel to the fiber; the radial axis is normal to the growth rings; and the tangential axis is perpendicular to the grain but tangential to the growth rings. The modulus of elasticity of wood in the longitudinal direction is generally 13 to 23 times greater than in either of the other two directions; compressive strength is about 9 times higher in the longitudinal direction than the other two. The ratios are based on testing of clear wood samples (*4*).

Clear wood has a surprisingly low coefficient of variability for strength properties. The coefficient of variability is generally from 20 to 25 percent. Clear defect-free timber has good strength and stiffness characteristics, and is highly predictable, in its longitudinal direction. However, designers and users must be aware of the problems that can develop because of various strength-reducing defects as well as wood's weakness in its other two axes.

Because of its growth process, single-direction strength properties, and susceptibility to moisture-induced strength reduction, wood has some inherent weaknesses. Knots, slope of grain, and high moisture content can substantially reduce both its strength and stiffness.

A knot is that portion of a branch that has grown into the trunk of the tree. A knot interrupts the grain continuity and changes the direction of the wood fibers. The strength properties are reduced because the clear wood is displaced by the knot and the fibers around the knot are distorted, causing cross grain. Knots have a greater effect on strength in axial tension and bending than in axial compression.

Sloped grain causes lines of stress to be only partially in the longitudinal fiber direction, thereby reducing strength properties. Moisture causes a reduction in the cementing strength of the lignin, which holds the cells together. Other localized defects such as pitch pockets and rot, along with knots, slope of grain, and moisture, will also reduce the strength properties. Moisture contents in excess of 19 percent, in most sawn-lumber species, or 16 percent, in glued laminated lumber, cause reductions in allowable design values.

The quantity and location of defects are limited by timber grading, but there is no feasible way to eliminate all defects in large sawn timber members. The presence of defects can substantially increase the coefficient of variability, and therefore reduce the allowable design values. The design values published by the National Forest Products Association (NFPA) are based on computations that use the strength levels of clear green wood provided by the Forest Products Laboratory. The ASTM methods used predict strength levels that are expected to be exceeded by 95 percent of clear wood tested. Standard ASTM reductions are made from these clear wood values to account for safety and duration of load. For individual grades, the resulting values are further reduced, in accordance with ASTM standards, to reflect the predicted effect of knots or other differences from clear wood permitted by each grade classification. This basically means that clear green wood strength properties, which are met by 95 percent of the samples, are reduced by a factor of 2.1 as a factor of safety and to account for duration of load. Further reductions are made in grading to account for defects. Higher moisture content further reduces allowable design values. Timber graded as "select structural" (SS) is basically defect free. This quality of material is becoming difficult to find and therefore expensive (*5*). Table 1 shows NFPA design values for Douglas fir structural joists and planks.

More precise and therefore higher design values for any particular single-member design may be obtained by using machine-stress-rated lumber. Machine-stress-rated lumber is actually tested to determine its modulus of elasticity. The remaining strength properties are then established by predetermined relationships between the modulus of elasticity and other strength properties.

PRESERVATIVE TREATMENT

A major problem encountered by users of timber throughout history has been premature deterioration of the wood because of fungal decay. In nature, wood decay is inevitable. If this were not the case, the forests would soon be cluttered with the remains of dead trees.

Fungal decay depends on adequate moisture content. The moisture content at which most species of wood will begin to decay is about 30 percent (or the fiber saturation point). Keeping bridges dry is seldom feasible. Many covered bridges were built in this country during colonial times. A waterproof roof structure was built over the bridge to keep it dry. These bridges were durable as long as the roofs were maintained. Generally, such bridges are no longer economically practical.

John Levy, a leading expert in preservative treatment at the Imperial College of Science and Technology in London, observed that "as far as the fungus is concerned, wood consists of a large number of conveniently oriented holes surrounded by food." Simply stated, preservative treatment is the addition of chemicals to the wood to make this "fungus food" toxic to the fungi.

Preserving wood through the application of chemicals is not new. Oils, tars, and pitches have been used by builders for thousands of years. Widespread use of preservative treatment developed with the expansion of the railway systems in Europe and this country during the 19th century. Preservative treatment creates a protective shell or envelope around the wood. This envelope of toxic material prevents the fungi from getting to the edible wood. Treatment with the proper preservative and process dramatically increases the useful life of wood exposed to the elements. The most effective means of applying the preservative chemical is with a pressure system, hence the term "pressure treatment." Proper pressure treatment will

TABLE 1 NFPA DESIGN VALUES FOR DOUGLAS FIR STRUCTURAL JOISTS AND PLANKS

Grade	Extreme Fiber Stress in Bending, F_b (psi)	Tension Parallel to Grain, F_t (psi)	Modulus of Elasticity, E (psi)
Select structural	1,800	1,200	1.8×10^6
No. 1/appearance	1,500	1,000	1.8×10^6
No. 2	1,250	650	1.7×10^6
No. 3/stud	725	375	1.5×10^6

generally increase the life of a timber product or structure by 5 to 10 times (*6*).

The effect of pressure treatment can be totally negated if the protective shell or envelope is broken. Therefore, wood should be completely fabricated before being treated, and drilling, cutting, or driving nails into treated wood should be avoided.

ENGINEERED TIMBER

In the past, there seemed to be a never-ending supply of high-quality, relatively defect-free timber. Economical design and use of timber seemed to be of minor importance. Today, although wood is a renewable resource, the sizes and quality of available timber are not what they used to be. In addition, the ever-increasing cost of construction products dictates more attention to efficient use of all structural materials, including wood. These circumstances necessitate better engineering design of wood structures.

Glulams

In the 1940s, the concept of gluing small select pieces of wood together to create a large structural timber was developed. These engineered structural timbers are generally referred to as "structural glued-laminated timbers" or "glulams."

Glulams are any members made from two or more layers of wood glued together with the grain of all layers approximately parallel. The laminations may vary as to species, number, size, shape, and thickness. The individual laminations cannot exceed 2 in. in thickness and are typically made from nominal 1- or 2-in. sawn lumber. Glulam timber is an engineered, machine-stress-rated product of a timber laminating plant, made of selected and prepared wood laminations glued together with waterproof adhesives.

An engineered timber product like glulam has many advantages over solid sawn timber. Smaller, easier-to-find, less-expensive pieces of wood are used. Large defects are totally removed and smaller defects are dispersed once they are combined into a larger structural unit. The effect of dispersing defects is illustrated by the reduction in the coefficient of variability for multiple laminate members. The coefficient of variability has been found to be 0.25 for 1 Douglas fir lamination, 0.15 for 4 laminations, 0.10 for 10 laminations and 0.08 for 16 laminations (*7*). Lower coefficients of variability result in more predictable, and therefore, higher allowable design values.

Lower-quality (and lower-cost) timber can be used in the low-stress areas of glulam beams, usually the center area near the neutral axis. This increases the efficiency of the timber usage and further lowers material costs.

Individual laminations can be finger-jointed with very little strength reduction, allowing fabrication of beams whose length is limited only by fabrication equipment, storage, and shipping.

Air-drying bridge timbers thicker than 2 in. requires 6 to 12 months—more time than lumber merchants can justify, because of high inventory costs. Even then, drying large timbers often causes excessive warping and checking. Kiln-drying is not practical for timbers more than 2 in. thick. Because glulams are made from nominal 1- or 2-in. laminations, air- or kiln-drying is practical and warpage can be controlled.

Glulams can be manufactured into many configurations, including curved girders and arches, giving bridge designers a wide selection of structural shapes with which to work. Glulam components are prefabricated at the laminating plant reducing field construction time. The efficiency of prefabrication usually saves money.

Laminated-Veneer Lumber

Laminated-veneer lumber (LVL) is a member of a larger group of wood-based composite lumber products called structural composite lumber (SCL). SCL is a composite of wood elements such as veneer sheets, wood strands or strips, and combinations thereof. The wood elements are glued with a waterproof adhesive to create a wide variety of products.

Laminated veneer lumber is made from thin sheets of wood glued together. Although LVL looks like plywood in cross section, it is structurally different. Plywood is cross laminated, with the grain of each layer running perpendicular to that of adjacent layers. In veneer lumber, the grain runs parallel in all of the layers. This produces a material that is extremely strong and reliable when a load is applied parallel to the grain. It is so reliable, in fact, that is was originally used to make airplane propellers during World War II.

As with glulams, LVL allows conversion of smaller-diameter, relatively low-grade logs into a wider and longer dimension of products. Veneers are not sawn pieces of lumber like the laminations used in glulams. Veneers are actually peeled from the circumference of the log. LVL uses the same wood in the log as solid-sawn, but the process of producing the wood elements and regluing them disperses the strength-reducing solid-wood characteristics even more so than for glulams. Because veneer lumber is made of overlapping 1/10- or 1/8-in.-thick layers of veneer, any defects that might have existed in the original wood are widely dispersed.

Solid sawn lumber may have knots, splits, or grain defects that go clear through the member. Glulams may have 3/4- or 1½-in.-deep defects, but laminated veneer lumber will only have defects that are 1/10 or 1/8 in. deep. The result is a finished product that is more uniform in quality, lower in variability, and therefore has more predictable and higher allowable design values. Laminated-veneer beams can theoretically be manufactured to any length. Current machinery, storage, and shipping limit the beam length to about 80 ft. The increased strength and predictability, combined with the unique sections that can be fabricated give the bridge designer much greater flexibility than has been available in the past. Structurally efficient I, tee, or box beam sections can be fabricated (*8*).

The process used to peel the veneers creates access for preservative penetration that is much better than that for solid wood or glulams. The veneer peeling process creates minute cracks perpendicular to the wood grain. The minute cracks (lathe checks) do not damage the wood fibers, but they allow excellent preservative penetration. Essentially, LVL allows full penetration of preservative treatment. Table 2 shows how design values are increased by the use of multiple pieces of wood, as in glulams or LVL.

TABLE 2 INCREASED DESIGN VALUES BY USING MULTIPLE PIECES OF WOOD AS IN GLULAMS OR LVL

Timber Type	Extreme Fiber Stress in Bending, F_b (psi)	Tension Parallel to Grain, F_t (psi)	Modulus of Elasticity, E (psi)
Douglas fir joists and planks			
Select structural grade	1,800	1,200	1.8×10^6
No. 2 grade	1,250	650	1.7×10^6
Douglas fir, glulam panel, Combination Symbol 5			
2 lams	1,800	1,600	2.0×10^6
3 lams	2,100	–	–
4 or more lams	2,400	–	–

TIMBER BRIDGE SYSTEMS IN THE FOREST SERVICE

Log and Solid Sawn-Timber Bridges

The first bridges built by the Forest Service were log superstructures on log crib abutments (Figure 1). Up until about 1945, short-span bridges in the Northern Region of the Forest Service were built almost exclusively of untreated logs. The abutments were log cribs and the superstructures were logs with transverse sawn timber decking. Many bridges constructed between 1945 and 1953 still had log crib abutments, but the beams in the superstructure were solid sawn timbers. Most of the sawn lumber was pressure treated to increase its resistance to decay. The transition from round logs to square sawn lumber made for better fit-up and faster installation.

FIGURE 1 Typical log bridge.

Treated superstructures lasted longer than untreated log crib abutments. In 1953, Forest Service bridge designers began designing and building pressure-treated, sawn-timber abutments to go with the superstructures. The bridges being built in the mid-1960s used beams as large as 12 in. wide by 26 in. deep and spanning up to 40 ft. A typical solid sawn-timber bridge is shown in Figure 2.

Glulam Beam Deck and Panel Bridges

In 1971, the Forest Service began using glulam beams as the main longitudinal load-carrying members in its bridges. Solid sawn beams were still used for the shorter-span bridges (less than 30 ft), but engineered structural members made possible higher allowable design values as well as deeper beams, which in turn made longer spans possible. The glulam beams used in the early 1970s ranged in size from 9 in. wide by 27 in. deep to 11 in. wide by 42 in. deep and spanned from 30 to 55 feet. Engineered timber had opened a whole new span range for timber bridges. These bridges used glulams only as the longitudinal beams. All other components of the bridge were still solid sawn timber.

Glulam beams proved to be so economical that in about 1974, most of the Forest Service stopped using solid sawn beams altogether. Virtually all timber bridges were being built with glulam beams. In 1974, the Forest Service also began using glulam deck panels placed transversely over the glulam beams. The glulam panel decks replaced nail-laminated decks that were constructed from edgewise 2- by 6-in. lumber nailed together. The individual nail-laminated pieces had been pressure treated, but the many nails penetrating the treatment envelope caused premature decay. The nail-laminated deck was also labor intensive at a time when labor costs were accel-

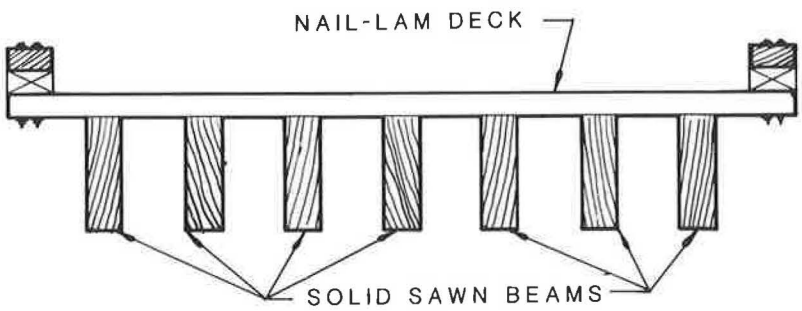

FIGURE 2 Typical solid sawn-timber bridge.

erating rapidly. Since 1974, virtually the entire bridge superstructure of all Forest Service timber bridges has been constructed using glulams. The Forest Service has built between 80 and 100 of these bridges in the Northern Region. The use of standardized components, such as glulam girders and panel deck systems, simplifies design and drafting details, and reduces field labor costs and jobsite construction time.

The panel deck system was originally developed with interconnecting steel dowels between the panels. This system created a very rigid, watertight deck that is still used by several regions of the Forest Service. The primary purpose of the dowels is to prevent differential movement of the deck panels that could cause cracking of a bituminous asphalt overlay. Because most of the bridges to be built in the Northern Region are not overlayed with asphalt, the use of dowels was discontinued in 1978. If a bridge constructed with undowelled deck panels is later paved, a geofabric material is placed between the deck panels and the overlay to reduce cracking.

Several methods of attaching the deck panels to the beams are available. The Forest Service typically nails the deck panels to the beams with 10- or 12-in. ring shank nails. The deck panels are drilled before treating. The beams are then drilled after the deck panels are placed across them. Because drilling breaks the preservative envelope, the holes are filled with liquid preservative before driving the nails. This reestablishes the protective envelope. Other systems, particularly the deck bracket developed by Weyerhaeuser Company, eliminate the need for nailing (Figure 3). This system should provide better durability.

The Forest Service has designed and constructed six glulam beam deck bridges in the last 2 years and the average cost for the superstructures was about $38 per square foot of bridge deck. Span lengths varied from 15 to 35 ft. Figure 4 shows a typical glulam beam deck bridge.

In 1977 Weyerhaeuser Company developed and tested a bridge system consisting of glulam panels set with the laminations vertical and spanning from abutment to abutment. The deck panels ran longitudinally instead of transversely and were designed to support the load without additional longitudinal support members. Transverse beams were placed under the deck panels to distribute the wheel loads to adjacent panels. These distributor beams allow the panels to be designed for less than a full wheel load and prevent the panels from moving independently of each other (*9*). See Figure 5.

This type of bridge has a shallow superstructure that, particularly in flat terrain, can be very desirable. A conventional timber bridge would generally require a roadway elevation several feet higher than a longitudinal glulam panel bridge. The structural inefficiency of the thin solid-slab cross section precludes this system for longer spans. A 10¾-in.-thick glulam panel will span about 25 ft. Using panels thicker than 10¾ in. is seldom cost effective.

The distributor beams developed by Weyerhaeuser Company were glulams. In the Northern Region of the Forest Service, steel channels are used to reduce the cross section of the distributor beams and to make installation easier.

The Forest Service has designed and constructed five longitudinal glulam panel bridges with distributor beams in the last year, and has been pleased with the appearance and ease of installation. The average cost for the superstructures of these bridges was about $36 per square foot of bridge deck. Span lengths varied from 15 to 27 ft. Figure 6 shows a typical longitudinal glulam panel bridge.

The Rocky Mountain Region of the Forest Service has used a similar system, but instead of distributor beams, the wheel loads are distributed by panels dowelled together with steel dowels. The dowel arrangement is similar to what is used on glulam beam deck systems. These bridges require more fabrication because of the dowel holes, and possibly more installation effort to insert the dowels. The final result has a clean look and there is nothing below the bridge to snag debris.

Stressed-Deck Bridges

The Forest Service has developed a slightly hybrid system for some of our temporary bridge needs. Many Forest Service access needs are short term. When the road system is no longer needed, the bridge is removed and the road obliterated. The Forest Service has designed a longitudinal glulam panel bridge that uses transverse bolts in place of distributor beams. The bolts are spaced at approximately 5-ft centers and the nuts are torqued to 200 ft-lb. The dowelling effect of the bolts is not sufficient to fully transfer wheel loads. Tightening the bolts creates friction between the deck panels that helps laterally transfer the wheel loads. Because wood has a tendency to compress when subjected to long-term loads, the panels are designed to fully support a single line of wheel loads. Therefore, the only need for the bolts is to prevent independent movement of the panels. The glulam panels are much lighter than the conventional portable temporary bridges and can generally be installed or removed by a small crew using light equipment. The supply cost of these bridges has been about $27 per square foot of bridge deck. Installation would add about $3 per square foot.

A system using unglued laminations compressed perpendicular to the lamination and the grain was developed in 1976 by the Ontario Ministry of Transportation and Communication in Canada. This system induces high interlaminar friction that, in effect, takes the place of the glue in a glulam. The concept was originally intended as a method for rehabilitating existing nail-laminated decks, but has been extended to new bridge decks as well. The system is similar to a longitudinal glulam panel deck except the individual laminations are held together by a compressive force imposed by posttensioning, high-strength steel rods. The rods are threaded through holes drilled in the center of the laminations at a spacing of 2 to 4.5 ft. These high-strength steel rods are perpendicular to the span direction and tensioned against steel bearing surfaces along the two outside edges of the bridge. When a bridge is fully tensioned, the friction between the individual laminations causes the entire bridge to function as an orthotropic plate.

This system has a number of disadvantages. The steel rods must be installed perpendicular to the laminations, which creates difficulties with any skewed bridge. The ends of the skewed bridge may have to be notched so that bearing plates can be installed. The system reduces prefabrication and generally increases field construction time. The high-strength steel rods and bearing plates are expensive and must be galvanized to protect against corrosion.

Wood has a tendency to compress slowly when subjected to long-term compressive forces. This compression of the wood

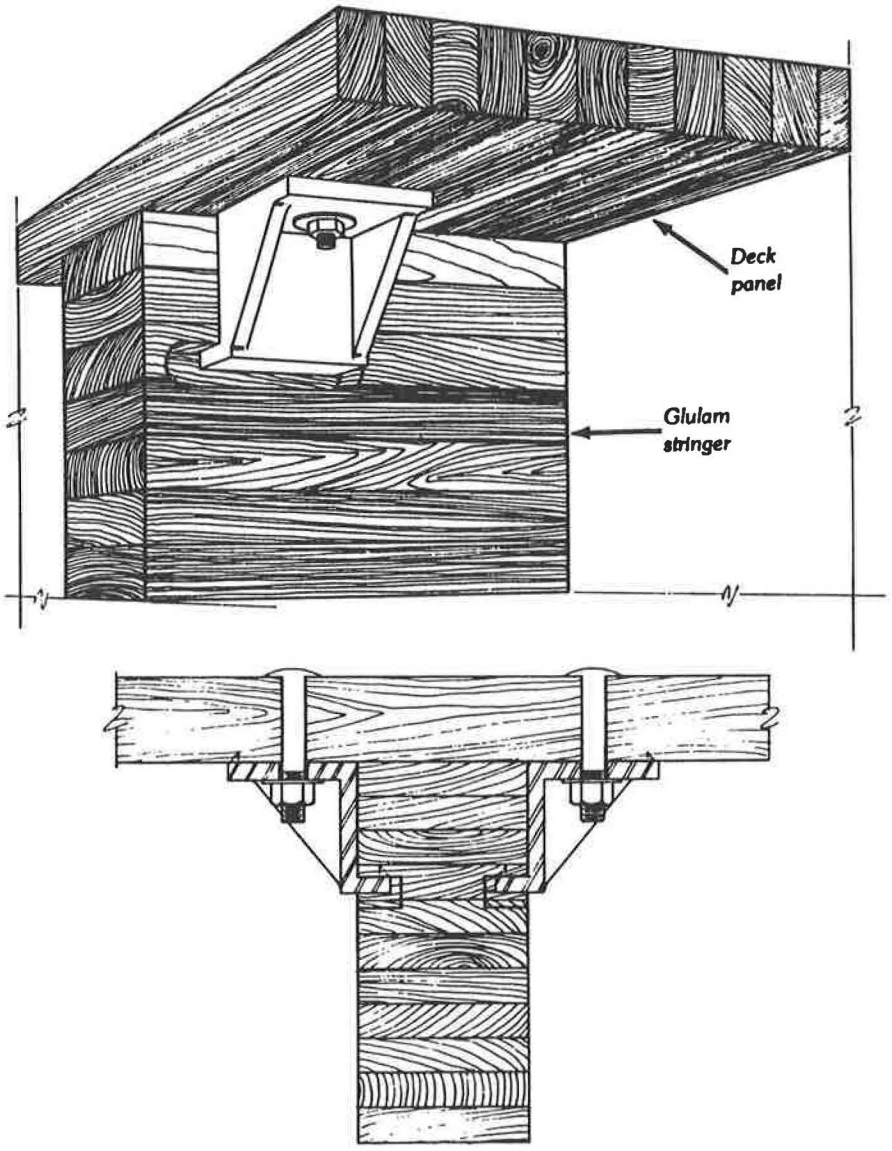

FIGURE 3 Glulam deck bracket.

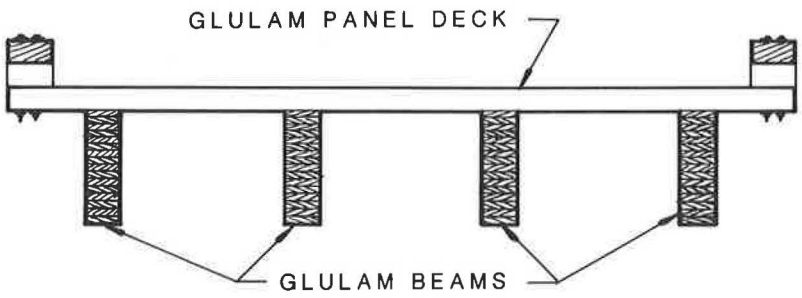

FIGURE 4 Glulam beam deck bridge.

FIGURE 5 Steel distributor beams.

reduces the force in the high-strength steel rods and thereby reduces the friction between the individual laminations. To counteract this effect, the steel rods must be retensioned several times. Testing has shown that the rate of compression decreases with time and most deformation has occurred within 6 months. Figure 7 shows a typical stressed-deck bridge.

Available cost data for 1988 and 1989 indicate that nine stressed-deck bridges were built nationwide by the Forest Service. The average cost for the superstructures of these bridges was about $75 per square foot of bridge deck.

A modification of this technology uses prefabricated LVL sections that are stressed together. The prefabrication reduces field time and cost substantially. Because LVL can be fabricated into a variety of shapes, structurally efficient tee or box sections can be used. The Forest Service has constructed two stressed LVL bridges in the Northern Region. One bridge

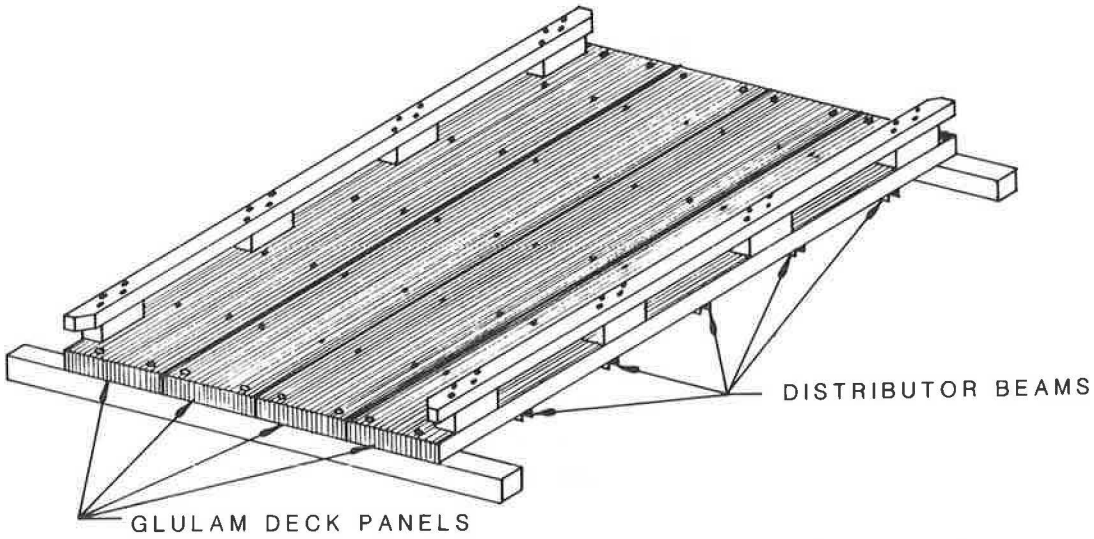

FIGURE 6 Two views of a longitudinal glulam panel bridge.

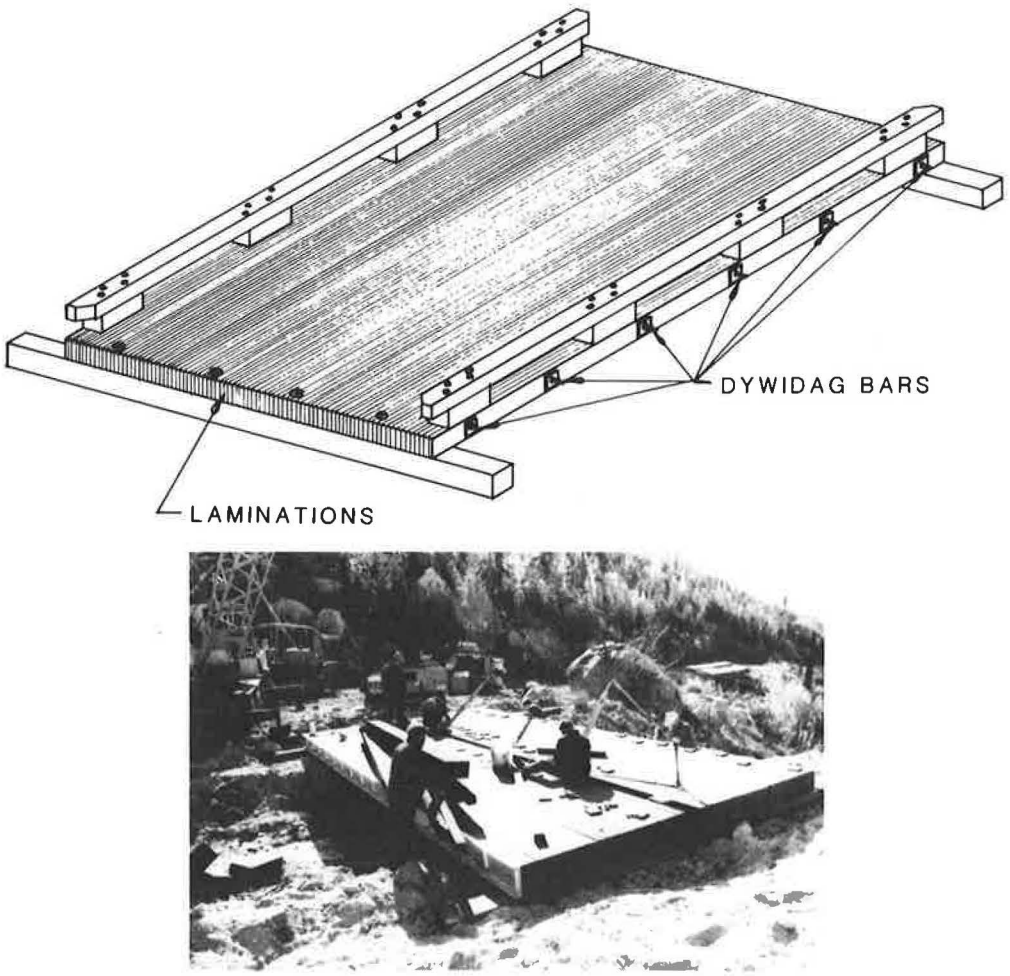

FIGURE 7 Two views of a typical stressed-deck bridge.

was a solid rectangular slab, 30 ft long. The other was a tee section, 38 ft long. Another stressed LVL tee section bridge was constructed in Nevada in 1989. The average cost for the superstructures of these three bridges was about $47 per square foot of bridge deck.

Costs of Concrete Versus Timber

Instead of a detailed comparison of timber versus concrete bridges, the following cost comparisons relate to short-span (less than 60 ft long) bridges in the Northern Region of the Forest Service.

Costs for short-span bridges are site-specific. The most economical 30-ft bridge at one site may be concrete, whereas at another site a 30-ft timber bridge may be preferred. Generally, easily accessible sites near concrete-prestressing plants can be most economically constructed using prestressed multibeam decks. Sites where concrete transportation costs will be high, such as remote rural areas, are often more economical for timber bridges. The lighter weight of timber makes it more easily transportable. The availability of cranes and the work area around the bridge site are also factors affecting the cost comparison. Concrete beams require heavy lifting equipment and an area for the equipment to maneuver. Timber beams and deck panels can often be set in place with a moderately sized backhoe or front-end loader.

Span length is probably the major factor in determining the cost-effectiveness of one bridge material over another. As the span length of a timber bridge increases, the depth of the timber beams or longitudinal glulam panels also increases. Increased material costs result in the cost per square foot increasing faster than the span length.

As the span of a concrete bridge increases, the concrete section used will remain the same throughout a certain span range. As an example, a 27-in.-deep section with three stems (commonly referred to as a "Trideck" beam) is used for spans from 30 to 60 ft. As the span increases, more prestress strands are added. Prestressing strand is relatively inexpensive. Therefore, because of economy of scale, the cost per square foot of a concrete bridge will often decrease as span lengths increase. The cost per square foot of concrete bridges will generally jump at span lengths at which deeper, more massive sections are needed.

Forest Service experience in the Northern Region has been that timber bridges are more economical than concrete bridges up to a span of about 28 ft. Beyond 28 feet, prestressed concrete multibeam bridges are generally more economical (Figure 8). This relationship changes slightly from year to

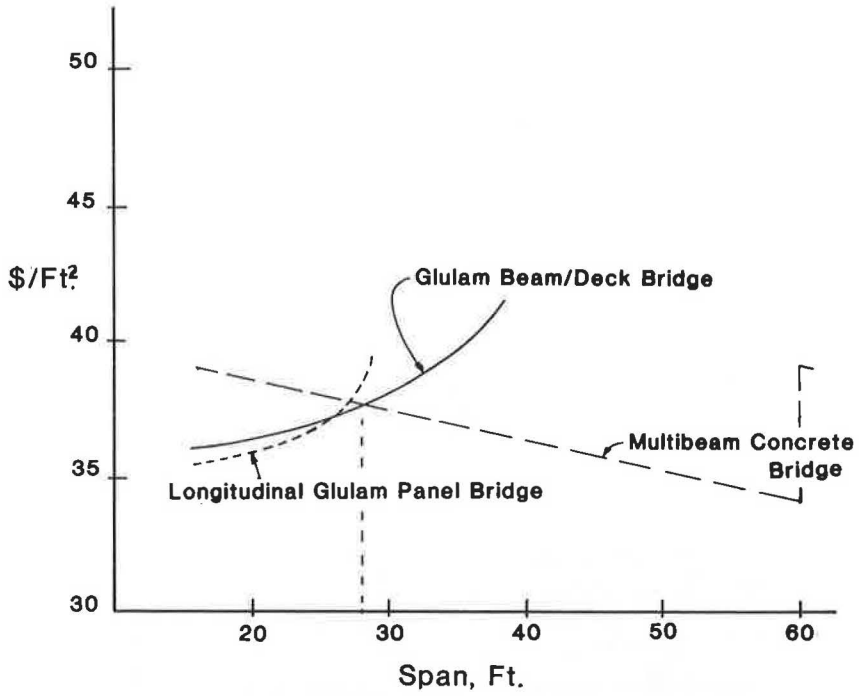

FIGURE 8 Cost comparison for timber and concrete bridges.

year, depending on timber and concrete material costs. As previously stated, location can also be a major variable.

SUMMARY

Technological advances in engineered wood products have made modern timber bridges a practical, reliable, and often economical solution to the problems encountered in replacing bridges on low-volume roads. Proper preservative treatments make wood resistant to decay. Engineered wood products, like glulam and LVL, allow use of smaller, defect-free pieces of wood. Designs that use prefabricated sections that are joined together at the bridge site reduce construction costs and overcome problems associated with using nails and field fabrication.

The performance characteristics of wood make a durable and reliable material for use in bridge construction. Wood is very resistant to impact loadings and damage from road salts. Timber bridges fit well into a natural setting and can also be an aesthetic addition to an urban environment.

Costs of glulam beam/deck bridges and glulam longitudinal panel bridges are consistently competitive in shorter spans, up to 30 ft, particularly on low-volume roads. Stressed deck systems using LVL sections show promise, particularly as timber prices increase.

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Reliability Analysis for Wood Bridges

ANDRZEJ S. NOWAK

A probabilistic approach for analysis of wood bridges is discussed. Load and resistance parameters are treated as random variables. The statistical models are derived on the basis of truck surveys, test data, and analysis. The mean maximum 75-year live load is calculated for single- and two-lane bridges. For low-volume roads, the maximum moments can be reduced by about 10 percent. Resistance is considered for timber stringers, glued-laminated (glulam) girders, and stressed decks. A considerable variation of modulus of rupture (MOR) is observed for sawn lumber. The degree of variation of MOR is considerably reduced in case of glulam girders and stressed decks. Reliability is a convenient measure of structural performance. Reliability indices are calculated for three structural types. The effect of traffic volume is negligible for timber stringers, but it can be considered for glulam girders and stressed decks.

Wood is an attractive material for bridge construction. It can be used successfully on low-volume roads. Very promising are new structural systems, including glued-laminated (glulam) girders and stressed decks. They allow for the use of low-grade local materials and are suitable for designs and repair or rehabilitation projects. However, there is a need for a rational basis for the development of design criteria.

Traditionally, a considerable variation of mechanical properties such as modulus of rupture (MOR) and modulus of elasticity (MOE) led to low values of the allowable stresses. Introduction of new technologies, including glulam girders and stressed decks, requires a new approach to the design and evaluation of wood structures. Load and resistance parameters are random variables. Therefore, reliability is a convenient measure of the structural performance that also provides a rational basis for comparison between wood and other structural materials.

The parameters involved in the design and evaluation of wood bridges are reviewed and a probabilistic basis for the development of load and resistance factors is developed. The load model is based on the available truck survey data. Resistance parameters are derived from test data and simulations. The procedure is formulated for calculation of reliability indices for sawn stringers, glulam girders and stressed decks, as shown in Figure 1.

BRIDGE LOADS

Bridge load components include dead load, live load, dynamic load, environmental loads and others (collision). A combination of the first three components is considered.

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Dead Load

In case of wood bridges, dead load is usually a small portion of the total load. The major components of dead load, D, are the weight of the structural members and the weight of asphalt (if any). The density of wood, which varies depending on species and moisture, is assumed equal to 60 lb/ft^3 for hardwood and 40 lb/ft^3 for softwood. Dead load is normally distributed. The statistical parameters of D are presented in Table 1 (*1*).

Live Load

Live load, L, covers a range of forces produced by vehicles moving on the bridge. The effect of live load depends on many parameters, including span length, truck weight, axle loads, axle configuration, position of the vehicle on the bridge (transverse and longitudinal), number of vehicles on the bridge (multiple presence), and stiffness of structural members. The maximum live load effect also depends on the time period considered. Bridges are usually designed for an economic life of 50 to 75 years. Shorter periods of time are considered in case of serviceability limit states.

The development of the live load model requires an extensive data base, weigh-in-motion (WIM) measurements, or truck surveys. The model described in the following paragraphs is based on the results of the truck survey performed by the Ontario Ministry of Transportation in 1975 (*2*). About 10,000 heavy trucks were measured (only heavily loaded vehicles were included).

Moments were calculated for various simple spans. The calculations were performed for trucks and axle configurations, with about 10 configurations per vehicle. For spans from 10 to 100 ft, the results are shown in Figure 2 on normal probability paper. The vertical scale is the inverse normal probability function. Any distribution represented by a straight line on normal probability paper is a normal function. Therefore, the shape of obtained distributions indicates the type. The moments are divided by the design values, for the HS-20 truck (*3*).

The maximum 75-year moment can be estimated by extrapolation. The surveyed trucks represent about 2 weeks' traffic on an Interstate highway. In 75 years, there will be about 1,500 times more vehicles. Let N be the number of trucks and axle configurations. For a 2-week period, N = 100,000, and for 75 years N = 150,000,000. The corresponding probability of occurrence, P_N, is equal to the inverse of the number of vehicles, $P_N = 1/N$. The corresponding inverse normal distribution, Z, is

$$Z = \Phi^{-1}(P_N) \quad (1)$$

where Φ is the standard normal distribution function.

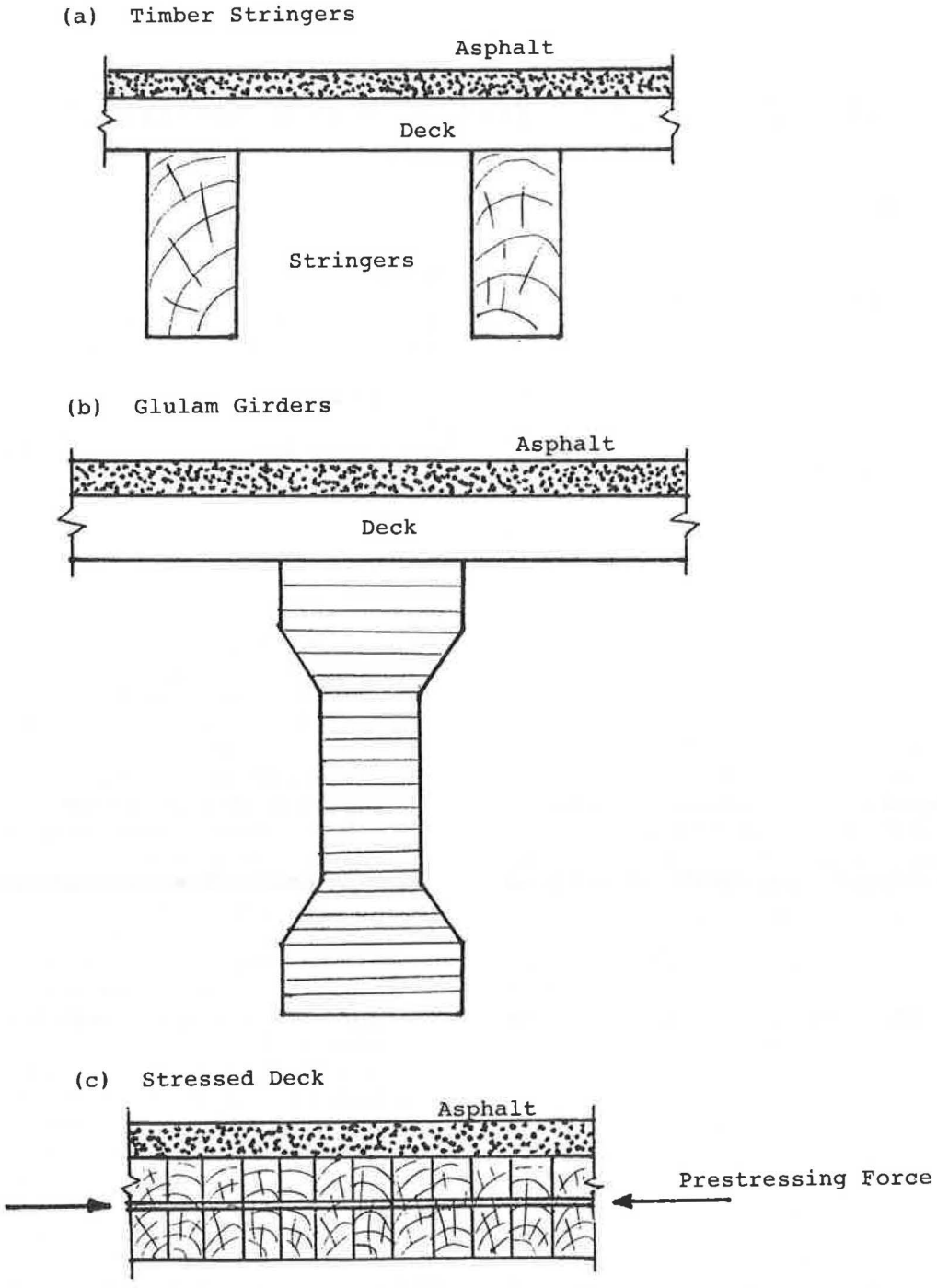

FIGURE 1 Wood bridge types considered in this study.

TABLE 1 STATISTICAL PARAMETERS OF DEAD LOAD

Item	Mean-to-Nominal	Coefficient of Variation
Wood Members	1.05	0.10
Asphalt	3.5 in	0.15

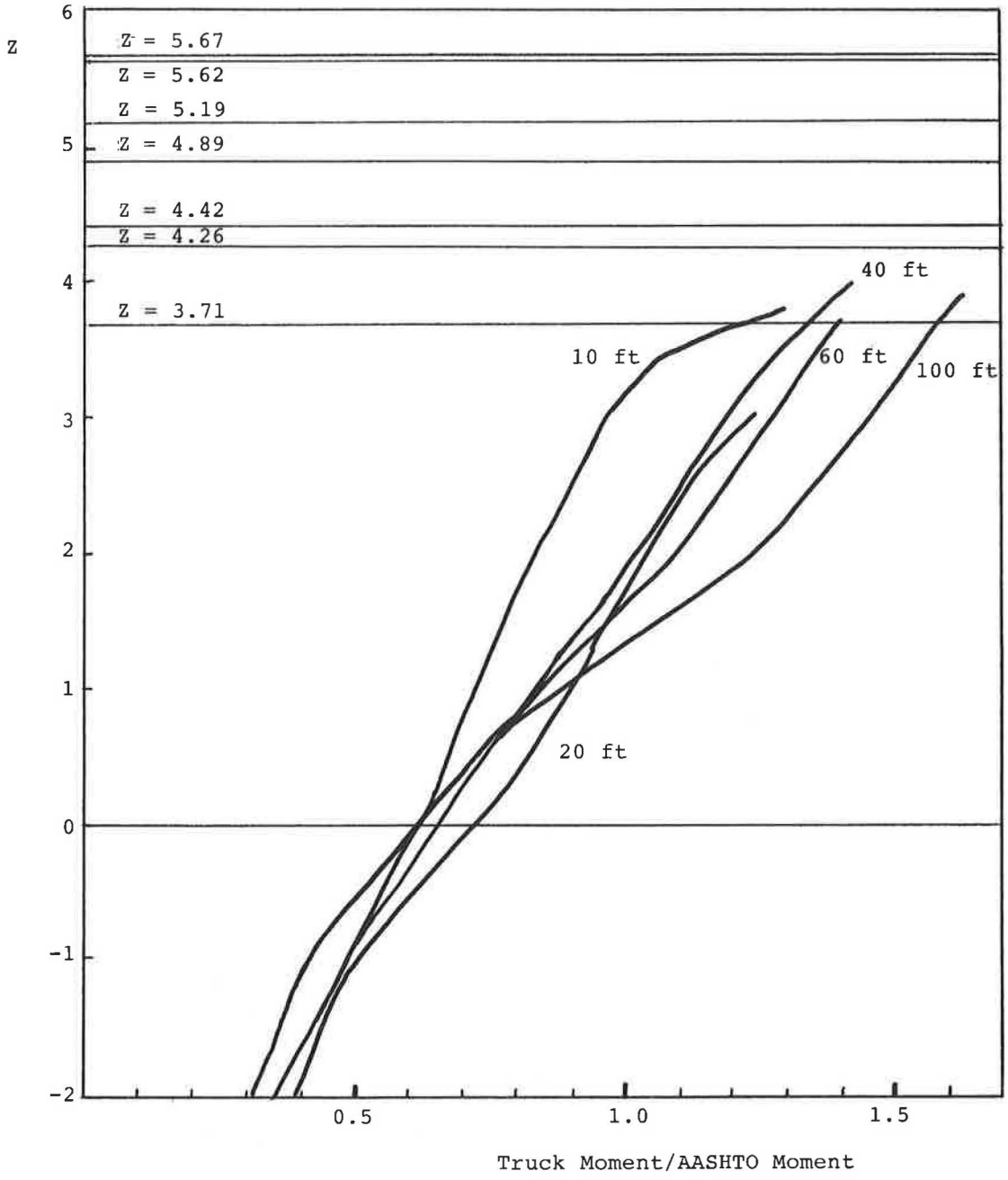

FIGURE 2 Distributions of moments on normal probability paper.

Values of time period, T, number of trucks and axle configurations, N, and corresponding probability and inverse normal distribution function are presented in Table 2.

The maximum moment is calculated for single- and two-lane bridges. For a single lane, one truck governs for short and medium spans (up to about 100 ft). For two lanes, the maximum effects are obtained for two trucks side-by-side, with perfectly correlated weights. The probability of occurrence of such an event is about 500 times less than occurrence of a single truck. If for a single truck $Z = 5.67$, then for two perfectly correlated trucks $Z = 4.50$, which corresponds to the maximum monthly truck.

The mean maximum 75-year moments for a single lane are plotted in Figure 3. For two lanes, the mean maximum moment is obtained for two trucks, one per lane, each about 0.85 of the mean maximum 75-year truck. For comparison, the design AASHTO (*3*) moments are also shown.

For low-volume roads, the number of trucks is lower. Depending on the actual average daily truck traffic (ADTT), the mean maximum 75-year moments are reduced compared to what is shown in Figure 3. The moment ratios are presented in Table 3. M_{high} is the moment calculated for a busy interstate highway; M_{low} is the moment for a reduced traffic volume. The corresponding values of Z are also included in the table.

TABLE 2 VALUES OF T, N, P_N, AND Z

Time Period	Number	Probability	Inverse Normal
T	N	P_N	Z
75 years	150,000,000	7_{10}^{-9}	5.67
50 years	100,000,000	1_{10}^{-8}	5.62
5 years	10,000,000	1_{10}^{-7}	5.19
1 year	2,000,000	5_{10}^{-7}	4.89
1 month	200,000	5_{10}^{-6}	4.42
2 weeks	100,000	1_{10}^{-5}	4.26
1 day	10,000	1_{10}^{-4}	3.71

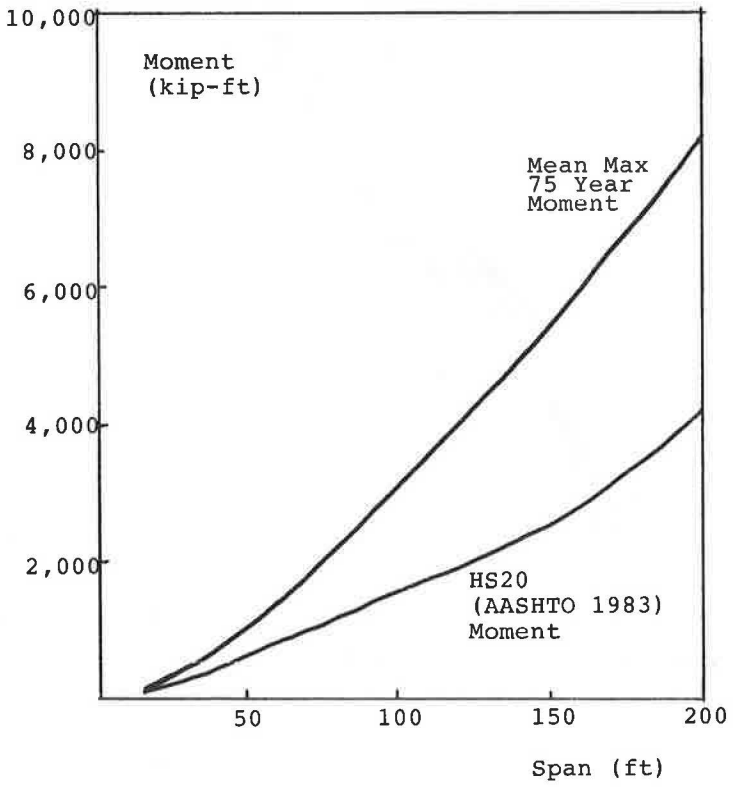

FIGURE 3 Mean maximum 75-year moments and AASHTO moments.

Dynamic Load

In the current AASHTO specification (*3*), there is no impact specified for wood bridges. However, observations indicate that dynamic loads should be considered. Extensive tests (*4*) and analytical simulations (*1*) provide a basis for the development of design provisions for steel and concrete girder bridges. Dynamic load is a function of three parameters: surface roughness, truck dynamics, and bridge properties (natural frequency). The calculated mean dynamic load is $0.15m_L$, and m_L is the mean maximum 75-year live load, and coefficient of variation is 0.80. However, very little data are avail-

TABLE 3 MAXIMUM LIVE LOAD FOR LOW-VOLUME ROAD BRIDGES

Traffic Ratio (High Volume ADTT/ Low Volume ADTT)	One Lane Z	One Lane M_{high}/M_{low}	Two Lanes Z	Two Lanes M_{high}/M_{low}
1	5.67	1.00	4.50	0.85
10	5.27	0.94	3.99	0.80
50	4.97	0.91	3.59	0.79
100	4.83	0.90	3.41	0.78

able for wood bridges. In this study, the mean dynamic load for wood bridges is assumed as $0.10m_L$ with coefficient of variation 0.80.

RESISTANCE

Bridge capacity depends on the resistance of its components and connections. The component resistance, R, is determined mostly by material strength and dimensions. In wood bridges, R depends also on moisture content and load duration. R is a random variable.

In this study, R is considered as a product of the nominal resistance, R_n, and three parameters: strength of material, M; fabrication (dimension) factor, F; and professional analysis factor, P.

$$R = R_n MFP \tag{2}$$

The mean value, m_R, of R and coefficient of variation, V_R, are given by the expressions

$$m_R = R_n m_M m_F m_P \tag{3}$$

$$V_R = \left(V_M^2 + V_F^2 + V_P^2\right)^{1/2} \tag{4}$$

where m_M, m_F, and m_P are the means, and V_M, V_F, and V_P are the coefficients of variation of M, F, and P, respectively.

Sawn Lumber

The variation in material strength and dimensions can be determined from the available test data. Extensive tests were carried out at the University of British Columbia and Western Forest Products Laboratory (*5*). The analysis of results was provided by Nowak (*6*). Statistical parameters were calculated for the MOR, MOE, dimensions, and correlation between MOR and MOE.

Examples of distribution functions of MOR are shown in Figure 4 for Douglas fir select and Figure 5 for Douglas fir Grades 1 and 2. Nominal dimensions are also indicated. The means and coefficients of variation are presented in Table 4 for Douglas fir (D-F), hemlock fir (H-F), and spruce-pine-fir (S-P-F). The allowable stresses are equal to about 0.25 to 0.35 of the mean MOR (*3*).

The statistical parameters were also calculated for MOE. The means and coefficients of variation are presented in Table 5. For comparison, the nominal (design) values of MOE vary from 1,600 to 1,800 ksi for D-F, 1,300 to 1,500 ksi for H-F, and 1,400 to 1,800 ksi for southern pine (*3*).

Typical scatter plots of MOR and MOE are shown in Figure 6 for D-F 2 by 8s and in Figure 7 for D-F 3 by 8s. Only a small degree of correlation between MOR and MOE is observed.

Observations of the test data indicate that dimension parameters do not depend on species. The nominal dimension is considered as dressed size (0.5 in. less than commercial size). The ratio of mean-to-normal varies from 0.96 to 1.025 and the coefficient of variation from 0.01 to 0.08.

For the professional analysis factor, it is assumed that $m_P = 1.0$ and $V_P = 0.10$.

Glulam Girders

The statistical parameters of MOR are derived on the basis of test data summarized by Ellingwood et al. (*7*) and presented in Table 6. Allowable stress specified by AASHTO (*3*) is 2,000 to 2,400 psi.

Stressed Wood

The resistance model is developed on the basis of tests carried out by Sexsmith et al. (*8*). Sawn timber 2 by 10 was placed in units 1, 2, and 3 ft wide. The units were transversely prestressed and loaded to failure. Three species were considered—hemlock fir, white pine, and red pine. The results of tests are plotted on normal probability paper in Figure 8 (*8,9*). The vertical scale is the inverse normal distribution function.

Load-carrying capacity of a prestressed unit can be calculated using the system reliability theory. The unit is a system of parallel elements (2 by 10) sharing the load. The system resistance, R_s, is a sum of element resistance values, R_e. Therefore, the mean of R_s values is

$$m_{R_s} = \Sigma_i m_{R_{ei}} \tag{5}$$

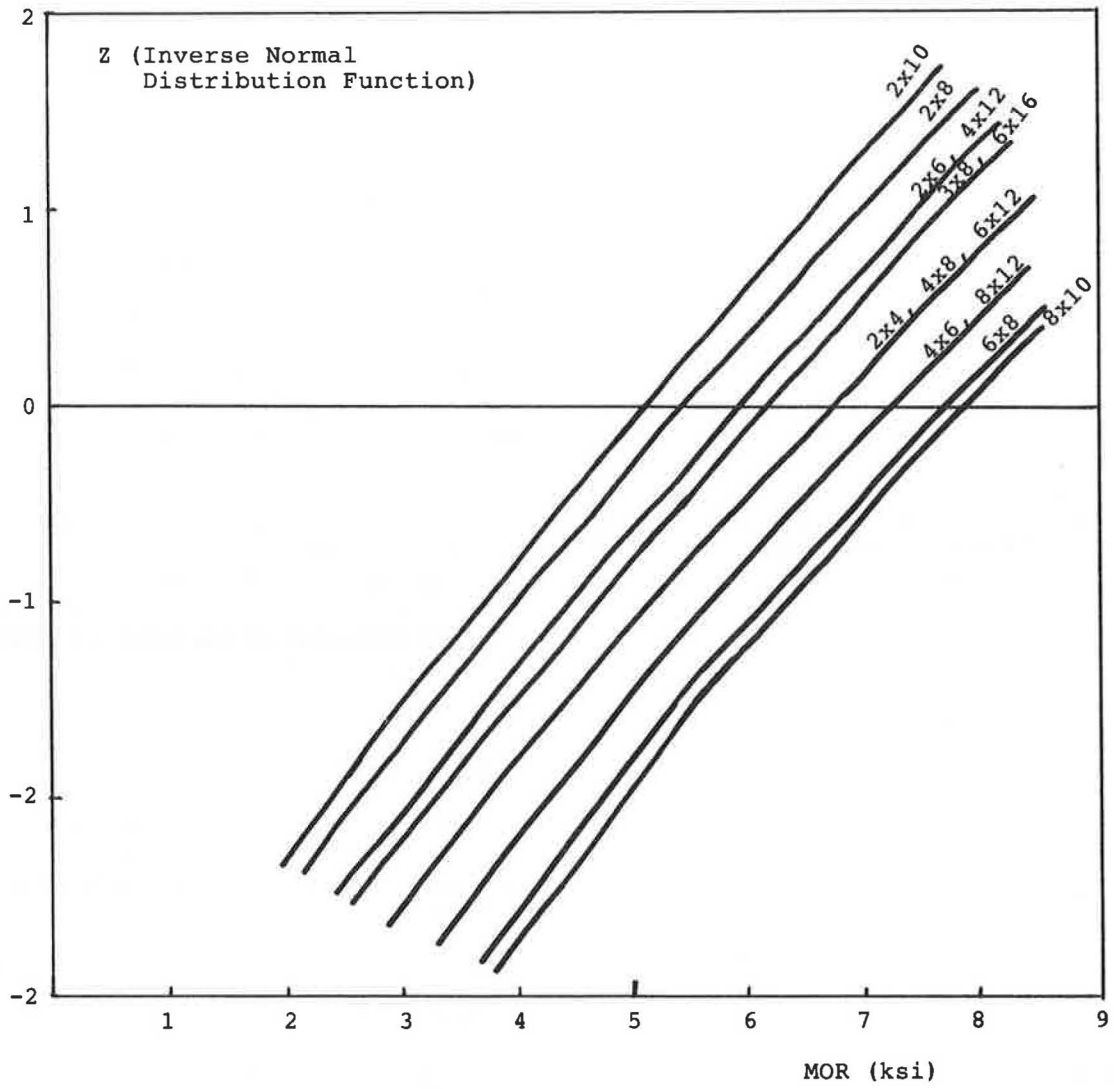

FIGURE 4 Cumulative distribution functions of MOR for D-F select on normal probability scale.

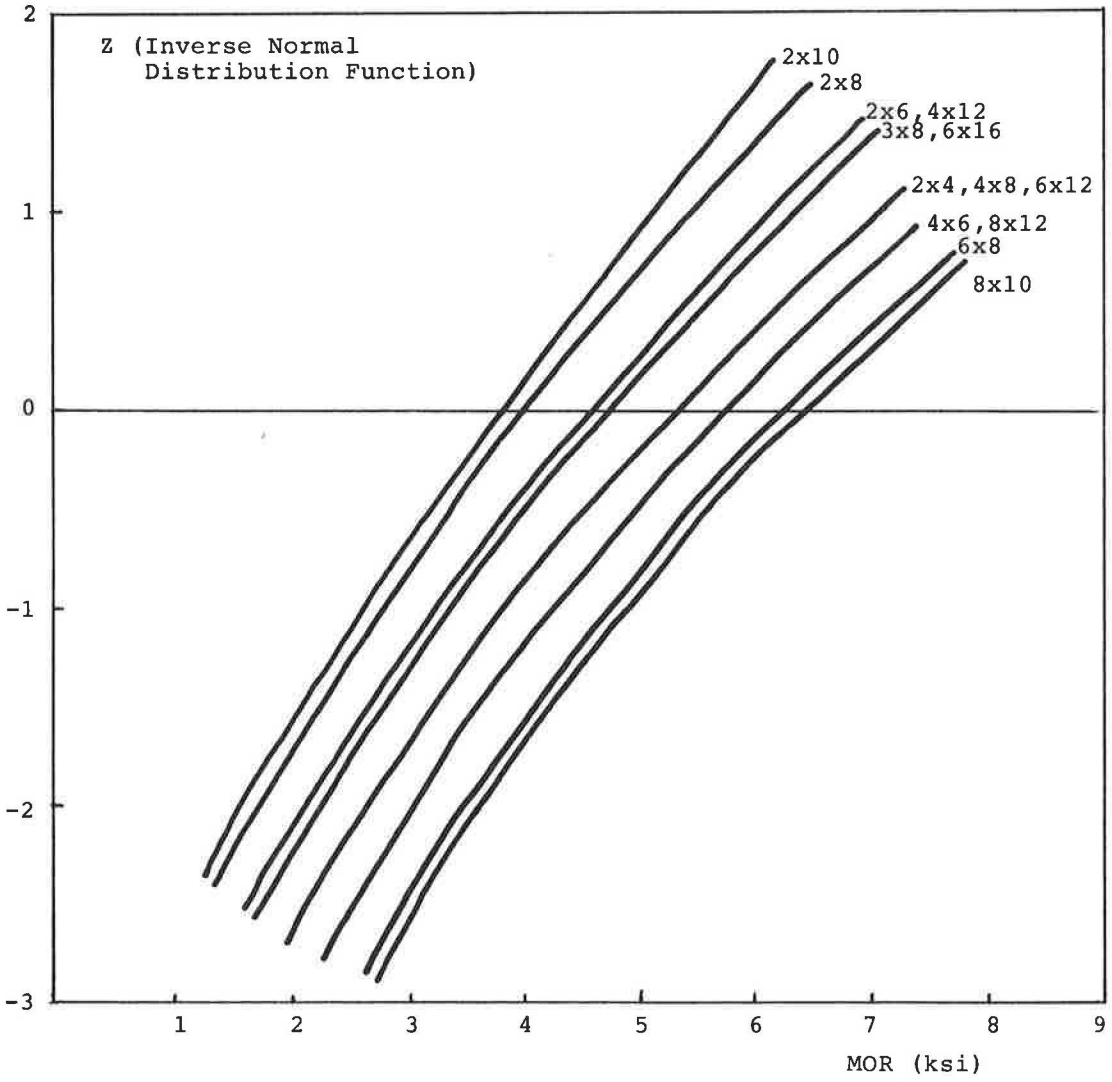

FIGURE 5 Cumulative distribution functions of MOR for D-F grades 1 and 2 on normal probability scale.

TABLE 4 STATISTICAL PARAMETERS OF MOR

Species	Size (in)	Select Mean (psi)	COV	Grade 1&2 Mean (psi)	COV
D-F	2x4	6800	0.22	5400	0.29
	2x6	6050	0.23	4600	0.30
	2x8	5500	0.25	4100	0.32
	2x10	5200	0.27	3850	0.32
	3x8	6200	0.23	4800	0.29
	4x6	7250	0.19	5750	0.26
	4x8	6800	0.22	5400	0.29
	4x12	6050	0.23	4600	0.30
	6x8	7800	0.17	6300	0.24
	6x12	6800	0.22	5400	0.29
	6x16	6200	0.23	4800	0.29
	8x10	7900	0.17	6400	0.23
	8x12	7250	0.19	5750	0.26
H-F	2x4	8450	0.26	7000	0.34
	2x6	7300	0.29	5450	0.34
	2x8	6600	0.30	4750	0.38
	2x10	6200	0.31	4350	0.38
	3x12	6600	0.30	4800	0.38
S-P-F	2x4	6950	0.32	6200	0.32
	2x6	6200	0.32	6500	0.34
	2x8	5400	0.30	4800	0.32
	2x10	4950	0.27	4500	0.33

TABLE 5 STATISTICAL PARAMETERS OF MOE

Species	Select Mean (ksi)	COV	Grade 1&2 Mean (ksi)	COV
D-F	1,500	0.20	1,300	0.21
H-F	1,650	0.18	1,500	0.20
S-P-F	1,500	0.17	1,400	0.21

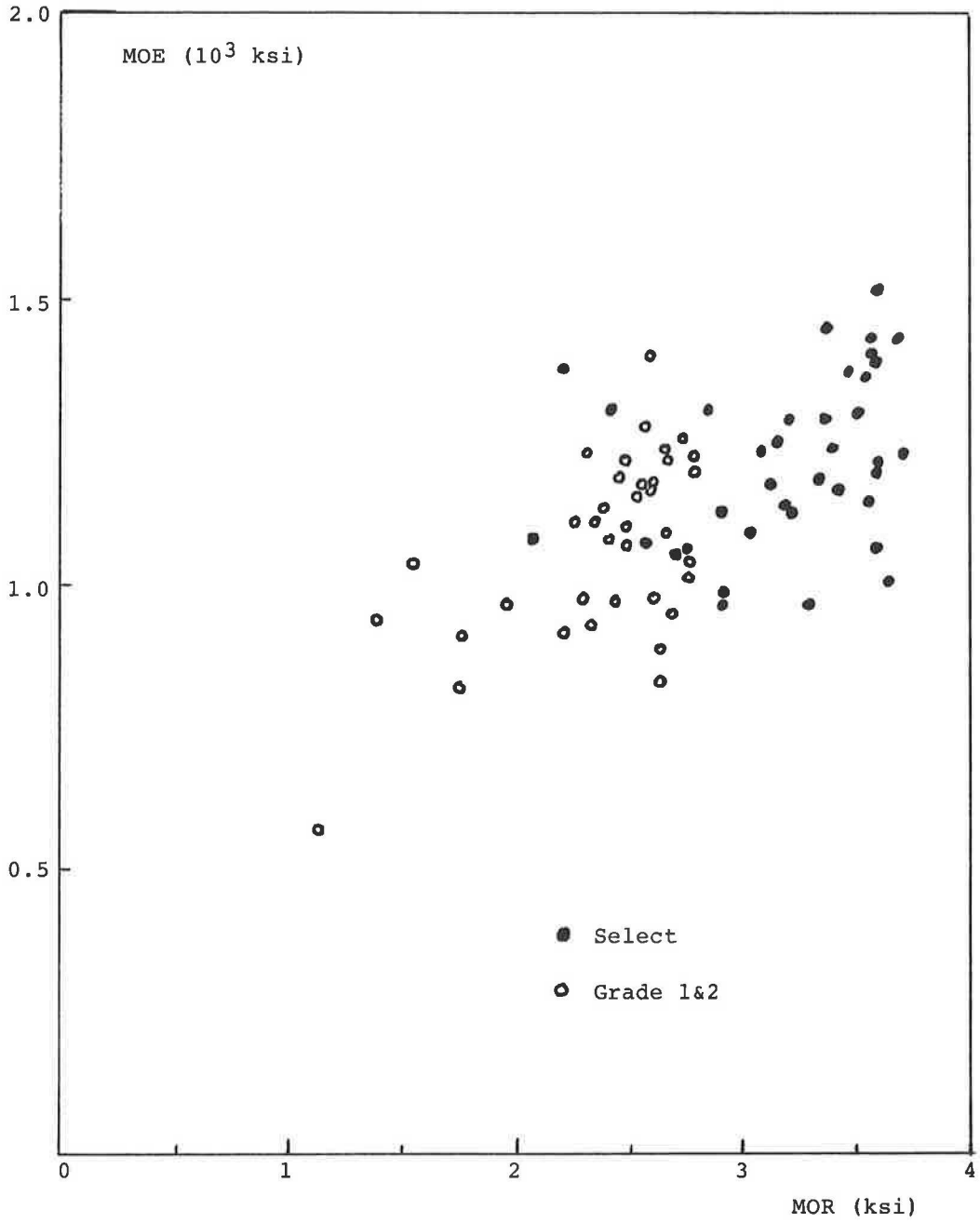

FIGURE 6 Scatter plot of MOR versus MOE for D-F size 2×8 (test results).

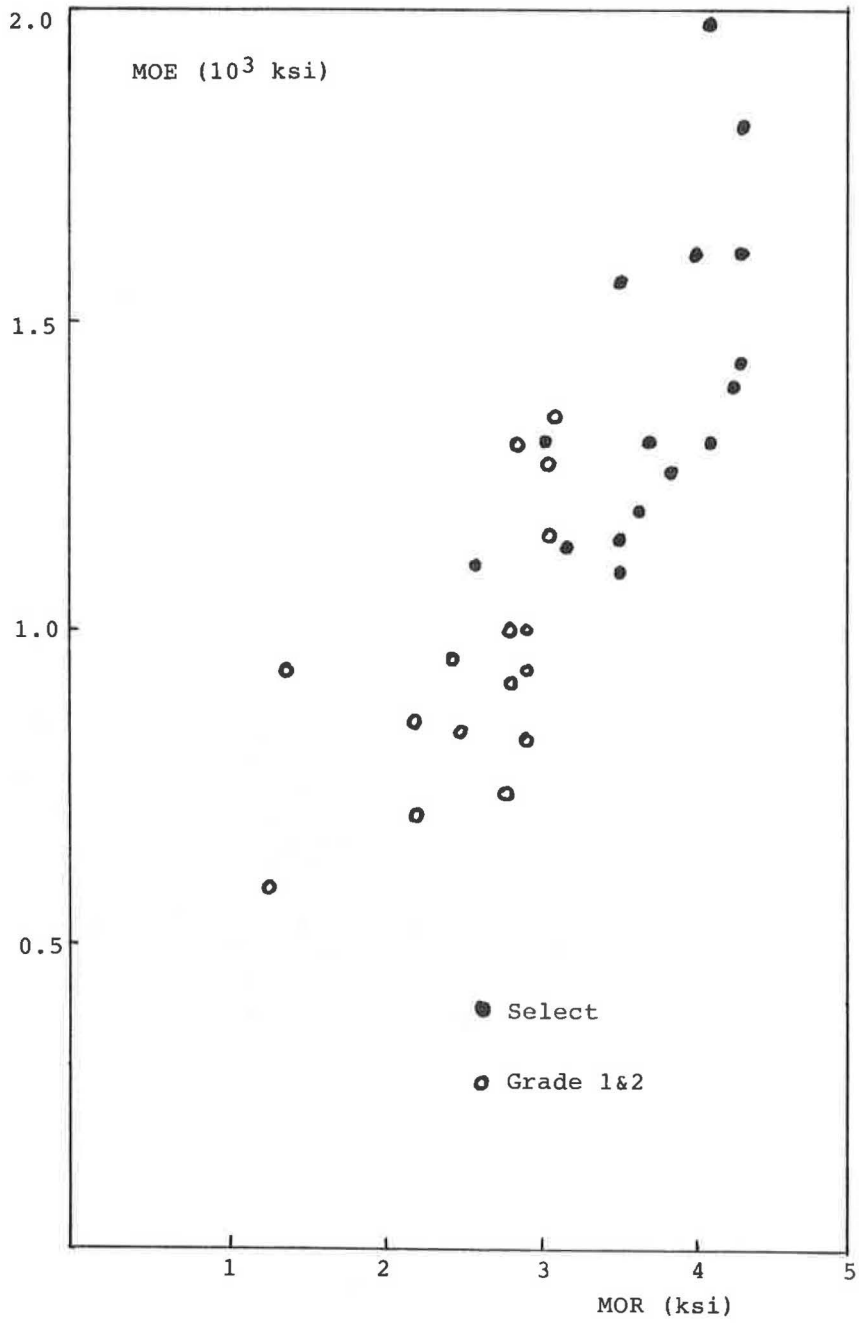

FIGURE 7 Scatter plot of MOR versus MOE for D-F size 3×8 (test results).

TABLE 6 STATISTICAL PARAMETERS OF MOR FOR GLULAM GIRDERS

Species	Size	Grade	Mean (psi)	COV
D-F	8x16	S 1	7070	0.11
	8x16	S 2	6240	0.16
Sitka Spruce	8x16	S 2	5040	0.33
W Hem	8x16	S 2	5300	0.20

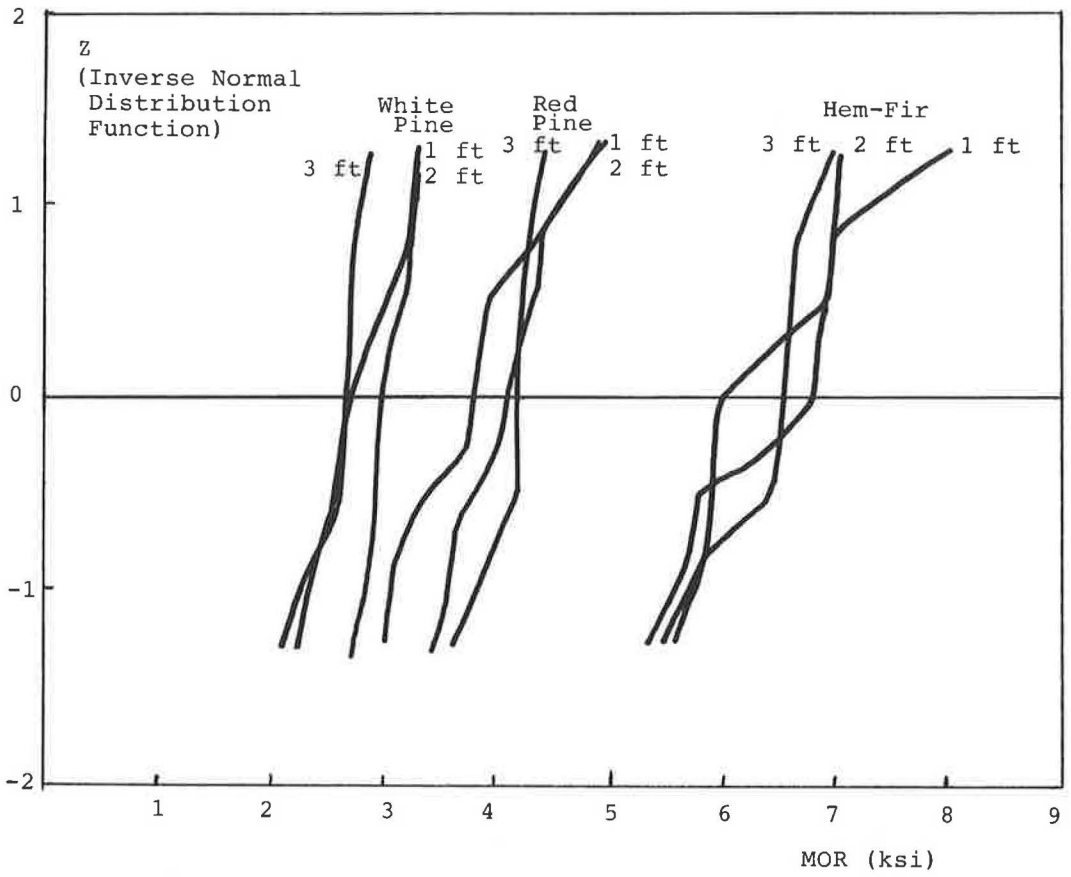

FIGURE 8 Cumulative distribution function of MOR for stressed wood on normal probability scale.

where m_{Rei} is the mean of R_e values for Element i. If all elements have the same distribution of R_e values, then

$$m_{R_s} = nm_{R_e} \tag{6}$$

where n is the number of elements in the system (unit).

For independent elements, the variance of the system is

$$s^2_{R_s} = \Sigma s^2_{Rei} \tag{7}$$

or

$$s^2_{R_s} = ns^2_{R_e} \tag{8}$$

where s_{Rei} = standard deviation of R_e for Element i.

The coefficient of variation of R_s is

$$V_{R_s} = s_{R_s}/m_{R_s} \tag{9}$$

From Equations 6 and 8, the coefficient of variation for a prestressed unit is

$$V_{R_s} = V_{R_e}/n^{1/2} \tag{10}$$

The calculated means, m_{R_s}, and coefficients of variation, V_{R_s}, are compared with the test results in Table 7 (*8–10*).

RELIABILITY ANALYSIS

The available reliability methods are described by Thoft-Christensen and Baker (*11*) and by Melchers (*12*), among others.

Limit states are boundaries between safety and failure. For example, a beam fails if the moment caused by loads, Q, exceeds the moment-carrying capacity, R. The corresponding limit state function, g, can be written

$$g = R - Q \tag{11}$$

If $g > 0$, the structure is safe; otherwise it fails. The probability of failure, P_F, is given by

$$P_F = \text{Prob}\,(R - Q < 0) \tag{12}$$

Let f_R and f_Q be the probability density functions (PDFs) of R and Q, respectively. Then $(R - Q)$ is also a random variable and it represents the safety margin, as shown in Figure 9. A direct calculation of P_F may be difficult, if not impossible. Therefore, it is convenient to measure structural safety in terms of a reliability index, β, defined as a function of P_F,

$$\beta = \Phi^{-1}(P_F) \tag{13}$$

where Φ^{-1} is the inverse standard normal distribution func-

TABLE 7 THEORETICAL AND OBSERVED PARAMETERS OF MOR FOR STRESSED UNITS

Unit width	Species	Mean Theory/Test	Coefficient of Variation Theory	Test
1 ft	Hem-Fir	0.98	0.13	0.16
	White Pine	0.82	0.15	0.15
	Red Pine	1.02	0.15	0.21
2 ft	Hem-Fir	1.05	0.09	0.09
	White Pine	0.75	0.10	0.07
	Red Pine	0.96	0.10	0.14
3 ft	Hem-Fir	1.02	0.09	0.08
	White Pine	0.87	0.08	0.08
	Red Pine	0.97	0.08	0.07

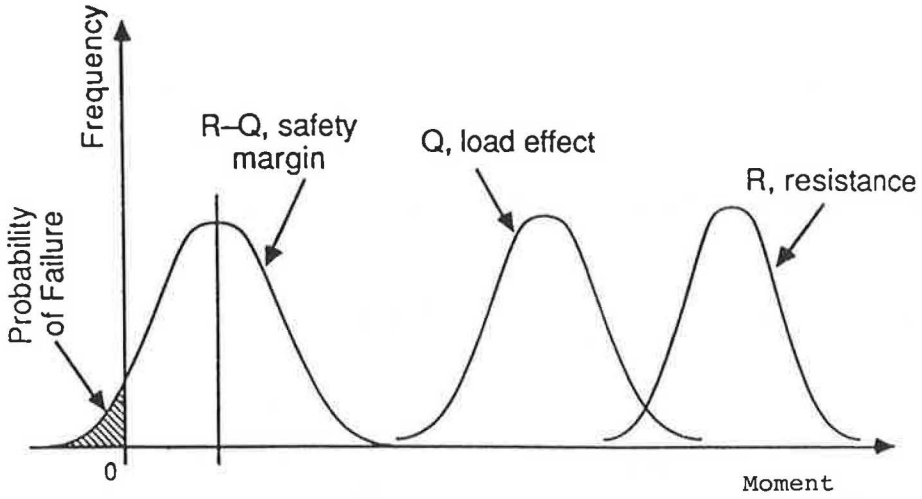

FIGURE 9 Density functions of R, Q, and $R - Q$.

tion. Examples of β's and corresponding P_F's are presented in Table 8.

There are various procedures available for calculation of β. these procedures vary with regard to accuracy, required input data, and computing costs. If both R and Q are independent normal random variables, then the reliability index is

$$\beta = \frac{m_R - m_Q}{\left(s_R^2 + s_Q^2\right)^{1/2}} \tag{14}$$

where m_R, m_Q are the means and s_R, s_Q are the standard deviations of R and Q, respectively.

In this study, R is treated as a lognormal random variable and Q is treated as a normal random variable. The reliability index is calculated using a special iterative procedure (*13*). A computer program was developed to facilitate these computations (*1*).

RELIABILITY INDICES

Reliability indices are calculated for three types of wood bridges: sawn stringers, glulam girders, and stressed decks.

Sawn Stringers

The design equation relates the allowable stress and loads:

$$R_{\text{allowable}}\, S = D + L \tag{15}$$

where D, L are moments of dead load and live load, respec-

TABLE 8 RELIABILITY INDEX AND PROBABILITY OF FAILURE

Reliability Index	Probability of Failure
1.0	0.159
2.0	0.0228
3.0	0.00135
4.0	0.0000317
5.0	0.000000287
6.0	0.000000000987

TABLE 9 RELIABILITY INDICES FOR WOOD BRIDGES

Type	Reliability Index	
	High Volume Road	Low Volume Road
Stringers	2.0-2.5	2.0-2.5
Glulam Girders	3.0-4.0	3.5-4.5
Stressed Deck	3.0-4.0	3.5-4.5

tively and S is elastic section modulus. For given D and L, the required $R_{\text{allowable}}$ S can be calculated.

The mean resistance is 3 to 4 times larger than $R_{\text{allowable}}$ S. The coefficient of variation of resistance is high (0.2 to 0.35 in most cases), because of variation in MOR. It dominates the reliability analysis. The reliability index for wood stringers is typically between 2.0 and 2.5. Traffic reduction on low-volume roads does not have any practical impact on the reliability level.

The reliability indices are higher (2.5 to 4.0) for structural systems consisting of stringers and the deck (*14*).

Glulam Girders

The parameters of R and Q in the limit state function are determined using the available statistical data. Mean resistance is 2.5 to 3.5 times larger than nominal (design) value. In most cases, the reliability indices are between 3 and 4.

For low-volume roads, the reliability indices are about 0.5 higher than for high-volume road bridges.

Stressed Wood Decks

Calculations are performed for a 1-ft-wide unit. There are no special provisions for stressed wood in the current AASHTO specification (*3*). Therefore, the nominal resistance is considered as for a nailed deck. The mean-to-nominal R is about 3 to 4 and coefficient of variation is 0.16. The reliability indices are between 3 and 4 in most cases.

For low-volume roads, the reliability indices are about 0.5 higher than for high-volume road bridges.

The calculated reliability indices are summarized in Table 9.

CONCLUSIONS

The approach is presented for reliability analysis of wood bridges. Statistical parameters of load and resistance are derived on the basis of truck surveys, test data, and analysis. Safety is measured in terms of the reliability index.

Reliability indices are calculated for three types of wood bridges: timber stringers, glulam girders, and stressed wood decks. The calculations are performed for high and low volumes of traffic. The results indicate that, for glulam girders and stressed decks, the current design provisions (*3*) are overconservative for low-volume road bridges. For timber stringers, because of a high variation of MOR, the volume of traffic does not have any practical effect on the reliability index.

The reliability-based approach can serve as a basis for the development of LRFD provisions for bridge design codes.

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Galvanized Cold-Formed Steel Bridges for Low-Volume Roads

R. W. LAUTENSLEGER AND I. P. ANDRADE

Typical installations, design practices, a wheel load distribution factor study, structural component tests, and a review of galvanized steel performance data are described for a galvanized cold-formed steel bridge system designed for low-volume roads. A number of these short-span bridges have been designed and built in Ecuador recently. Box sections used as longitudinal girders were shown analytically to provide better lateral distribution of wheel loads on bridge plank decks than conventional wide-flange beams or I-girders. Effective torsional stiffness of the box girders used in the study was determined from laboratory tests on two full-scale, 11.7-m-span, prototype box girders. Flexural tests were also conducted on the prototype girders. One girder was spliced by a field welding procedure and the other by a bolted splice design. Test results confirmed that strength and stiffness can be accurately predicted using state-of-the-art cold-formed steel design technology with some restrictions on geometry and welds. Durability performance data and life-cycle costing analyses on galvanized steel bridges in the United States indicate that the subject bridge system offers a durable, maintenance-free, and economical system when site environmental conditions are suitable for galvanized coatings.

Developing countries are seeking economical, easily erected, maintenance-free and durable bridges for their low-volume roads. Developed countries have similar needs for replacement or rehabilitation of aging low-volume bridges. GangaRao et al. (*1*) recently applied value engineering analysis techniques to low-volume bridges in the United States and concluded that the following parameters, with a weighting factor assigned, are the most important for selecting a low-volume bridge: material cost (23 percent), maintenance aspects (18 percent), durability (16 percent), service life (15 percent), availability (15 percent), and ease of construction (13 percent).

Applications, analysis, and component testing of an all-galvanized cold-formed steel bridge system designed for low-volume roads in Ecuador and other developing countries are described. In these countries, the hot-rolled steel structural shapes commonly used in developed countries for main structural members are not domestically produced and are expensive to import. Consequently, cold-formed sections of more readily available sheet and strip steel are often used for primary members such as longitudinal girders. Durability of the steel is enhanced by hot-dip galvanizing, which is available in many developing countries. Durability performance data and a life cycle cost study from the literature are reviewed.

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ALL-STEEL BRIDGES

For nearly 45 years, an all-steel bridge concept has been used for low-volume road bridges in the United States. The concept is based on the use of economical, mass-produced steel components: corrugated decking (bridge plank), hot-rolled wide-flange beam stringers (main longitudinal members), hot-rolled H-piles or cold-formed pipe piles and sheeting or bridge plank for substructures, and cold-formed guard rail for the safety railing. The individual steel components are relatively lightweight, making heavy lifting equipment unnecessary for erecting purposes, and are easily installed. Normally, except for a welder, no special skilled labor is required to construct these bridges. Except for the stringers and H-piles, the components typically have been hot-dip galvanized. The bridge plank deck is normally surfaced with a bituminous asphalt pavement after installation.

The components of the Ecuador bridge system (Figure 1) are similar to those of the U.S. system except for the primary structural members. Box sections similar to the prototype girder shown in Figure 2 replace the hot-rolled shapes used in U.S. bridges. They are fabricated by welding together two stiffened cold-formed channels. The channels are typically press brake-formed, but could be roll-formed. Shear and bearing stiffeners are welded inside before the channels are welded together with a lap between sections. The height-to-width ratio of the box sections varies, but typically is about 3 for longitudinal girders. Minimum thickness is 4.2 mm. The steel grade is equivalent to ASTM A36 or ASTM A570 Grade 36, depending on thickness. Cover plates are welded to the top and bottom flanges when required for bending strength and stiffness.

A rigorous welding quality control program based on American Welding Society standards for structural steel (D1.1) and sheet steel (D1.3) is used for all shop and field structural welding. All components are hot-dip galvanized following ASTM A123 or ASTM 153 specifications to provide a minimum of 610 g/m^2 coating weight (86 μm for steel thickness less than 6.35 mm, 99 μm for greater steel thickness). The decision to use galvanized components was influenced by the excellent performance of galvanized bridges in the United States.

Because of galvanizing-tank size limitations, sections are hot-dip galvanized in lengths not exceeding about 6 m. For the longer longitudinal girders, the galvanized coating is removed from the mating ends and the sections are joined by full-penetration groove welds. Zinc-rich paint is used to coat the splice welds. Box girders for bridge lengths to 28 m have been fabricated in this manner. A bolted splice design has

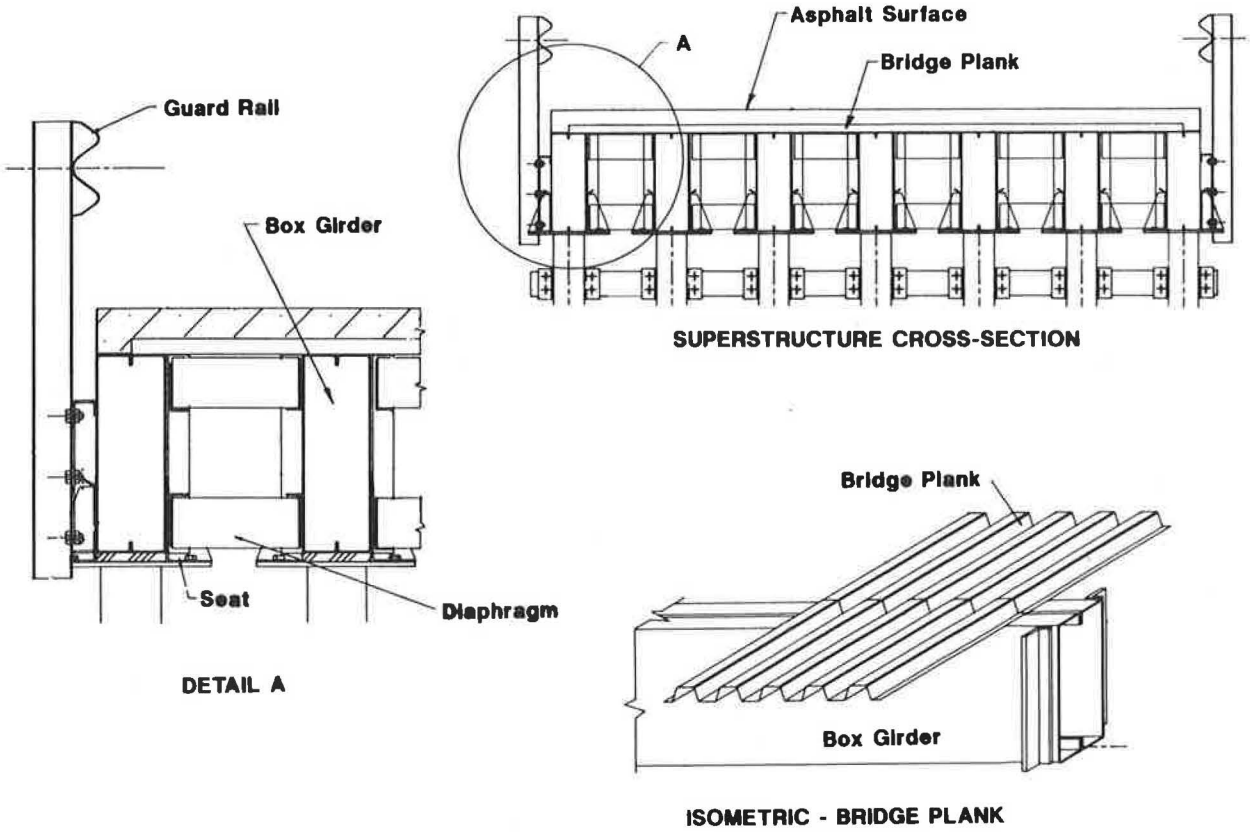


FIGURE 1 Cold-formed steel bridge details.

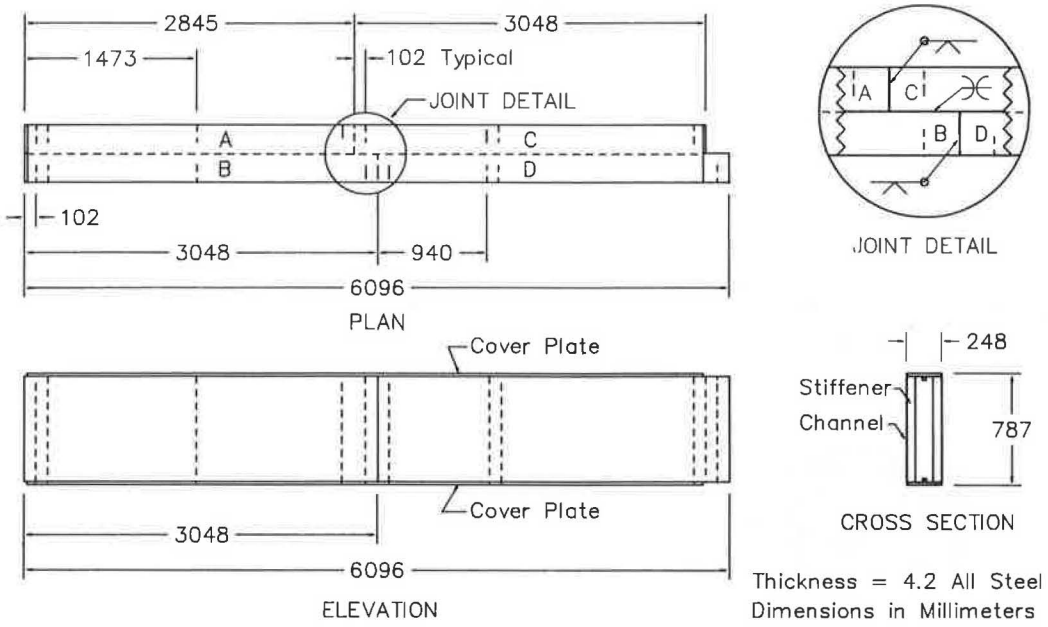


FIGURE 2 Prototype box girder test section.

also been developed and tested for use at sites where field welding is impractical.

Box girders have several distinct advantages over wide flange beams and I-girders in all-steel bridges. Less bracing is required for lateral stability of the compression flange, and lateral distribution of wheel loads is improved because of the high torsional stiffness of box sections. The double webs provide more uniform support for the bridge plank deck, plus more redundancy in the system, and the problem in I-girder bridges of the bridge plank effective span increasing under wheel load because of girder flange rotation is eliminated. The principal disadvantage of the box section is that it is not as efficient in vertical plane bending as the I-section.

GALVANIZED COLD-FORMED STEEL BRIDGE INSTALLATIONS

Ten galvanized cold-formed steel bridges have been locally fabricated and built on low-volume roads in Ecuador since 1987. Brief descriptions of two of the bridges follow.

Figure 3 shows the Colorado Bridge in the state of Chone, built in 1988. This 12.8-m-long bridge is designed to carry two lanes of traffic on a 7.0-m-wide deck supported by 10 longitudinal box girders. The girders are 780 mm deep by 250 mm wide and are fabricated from 4.2-mm-thick steel. The design was made for AASHTO HS20–44 truck loading. The abutment breastwalls are of tie-back anchored sheet piling. The abutment seat beams are cold-formed box sections supported by six hexagonal cold-formed steel columns mounted on a reinforced concrete pedestal footing.

Figure 4 shows the Zapallo Bridge in Chone, built in 1989. A delta truss substructure was used to shorten the effective span of this 26-m-long, 4.3-m-wide, single-lane bridge. The deck is supported by seven continuous box girders. The girders, with a 17.0-m center span, are about 650 mm deep by 230 mm wide and are fabricated from 5.5-mm-thick steel. The design loading was again AASHTO HS20–44. The vertical and inclined members of the 6.0-m-high delta trusses are fabricated box sections; the horizontal members are cold-formed channels. The trusses are mounted on a reinforced concrete pedestal footing. The sheet piling abutment breastwalls are tied back to deadman anchors at several levels.

FIGURE 3 Colorado Bridge, Ecuador.

FIGURE 4 Zapallo Bridge, Ecuador.

EVALUATION OF CURRENT LOW-VOLUME BRIDGE DESIGN PRACTICES

Low-volume bridges in the United States are currently designed using the same AASHTO specifications (*2*) as urban and Interstate highway bridges. Therefore, many low-volume bridge designs are believed to be overly conservative. The AASHTO specifications are also used for the Ecuador bridges, but are supplemented with the cold-formed steel structural design requirements for the American Iron and Steel Institute (AISI) specification (*3*) adjusted to AASHTO factors of safety.

GangaRao and Zelina (*4*) reviewed the AASHTO specifications recently to identify criteria that may be overly conservative or irrelevant for low-volume bridges. In addition to several geometric and functional criteria, they suggested that current fatigue and deflection requirements for structural design are excessively conservative.

For fatigue, they concluded that the current lowest level of AASHTO design cycles (100,000) used for low-volume bridges corresponds to an average daily truck traffic (ADTT) of 125 in one direction for a 50-year service life. Their conclusion was based on an observation that one AASHTO design cycle equals about 23 actual vehicle load cycles. They also concluded that an ADTT of 125 is about 10 times too high for typical U.S. low-volume roads and suggested that fatigue design be neglected entirely for low-volume bridges. However, fatigue checks have been used for the Ecuador bridges because of their lighter-gauge steel design.

For deflection, GangaRao and Zelina suggested that the live-load deflection criterion could be reduced from the current $L/800$ (L = span) limit to about $L/360$. A deflection limit of $L/500$ has been used for the Ecuador bridge design.

WHEEL LOAD DISTRIBUTION FACTORS

Another AASHTO design criterion may be overly conservative with regard to the Ecuador low-volume bridge designs: the wheel load distribution factor (DF) for longitudinal girders. In order to investigate this, an analytical study was undertaken. Finite-element elastic analysis techniques were used to include the structural interaction between the bridge plank deck, the longitudinal box girders, and the crossframes tying

the girders together. The ANSYS (5) finite-element code was used for the models.

Three-dimensional orthogonal grid models (Figure 5) were developed and analyzed for 12.2- and 24.4-m spans, each with one- and two-lane deck widths. Girder spacing, controlled by the lightest bridge plank allowable span, was set at 780-mm centers. The box girders were modeled as a series of three-dimensional beam elements with an effective torsional stiffness determined by tests described in the following section. For comparison, standard hot-rolled wide-flange beams were modeled in the same manner.

The bridge plank deck was modeled as a series of three-dimensional beam elements also. The element lines were spaced 0.61 m apart, transverse to the girders, and equal to the width of individual bridge planks. At crossing point nodes, the plank elements were linked to the girder elements.

Crossframe bracing effects were simulated by applying displacement boundary conditions at selected locations. Boundary condition locations are shown by arrowheads in Figure 5.

The analysis model girders were designed for AASHTO HS20–44 truck loads. The wheel loads were placed as concentrated loads to produce maximum bending moments in the longitudinal girders.

On the basis of results of the analyses and torsional tests, new DF equations were recommended for bridge-plank-on-box-girder bridges. The recommended equations are compared with the current AASHTO (2) equations as follows (S = center-center spacing of girders in meters). Numerical results from the analysis models are compared with both sets of equations in Figure 6.

	Distribution Factor	
No. of Traffic Lanes	*Current AASHTO*	*Box Girder Recommendation*
1	$S/1.676$	$S/1.753$
2 or more	$S/1.372$	$S/1.600$

The recommended DF equations reduce the single-girder design load by about 4 to 14 percent. In order to use the equations, crossframe or other external diaphragm bracing must be used between box girders at end reactions and at intermediate points such that the spacing does not exceed 10 m. Also, internal plate diaphragms must be used inside the girders at the end reactions, and the bridge plank must be continuous across the deck width.

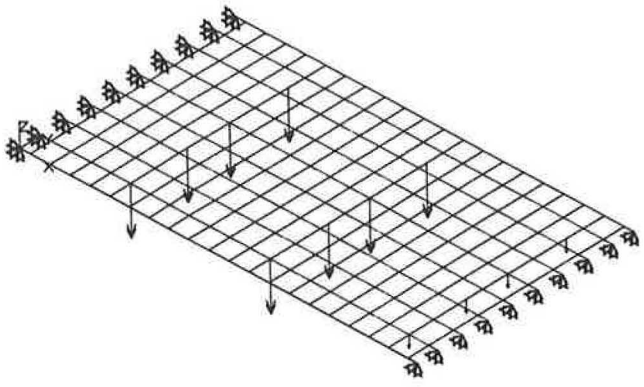


FIGURE 5 Typical DF grid model, 12.2-m span, two lanes, wheel load Case 2.

BOX GIRDER TORSIONAL AND FLEXURAL TESTS

Because no prior applications of cold-formed box sections to bridge girders were known, a testing program was developed to confirm design assumptions. The program included the following:

1. Fabrication of two prototype 12-m-long box girders—one with a bolted splice and one with a field-welded splice at midlength;
2. Torsional loading tests to determine effective torsional stiffness of the girders under several bracing conditions; and
3. Flexural loading tests to check flexural stiffness and ultimate moment capacity.

The tests were conducted at the Armco Research and Technology laboratory, Middletown, Ohio.

Four half-length girders (Figure 2) were designed and fabricated from 4.2-mm ASTM A569 steel with 248-MPa yield point. The box section was proportioned with the cover plate width-to-thickness ratio set at the maximum of 60 permitted by the AISI specifications. The ratio of web depth to thickness was set at 180, greater than the 170 maximum permitted by AASHTO but well below the maximum of 300 permitted by AISI for reinforced webs. The web stiffeners were sized according to AISI transverse stiffener criteria; the spacing was calculated to meet test requirements and was greater than that used for production girders.

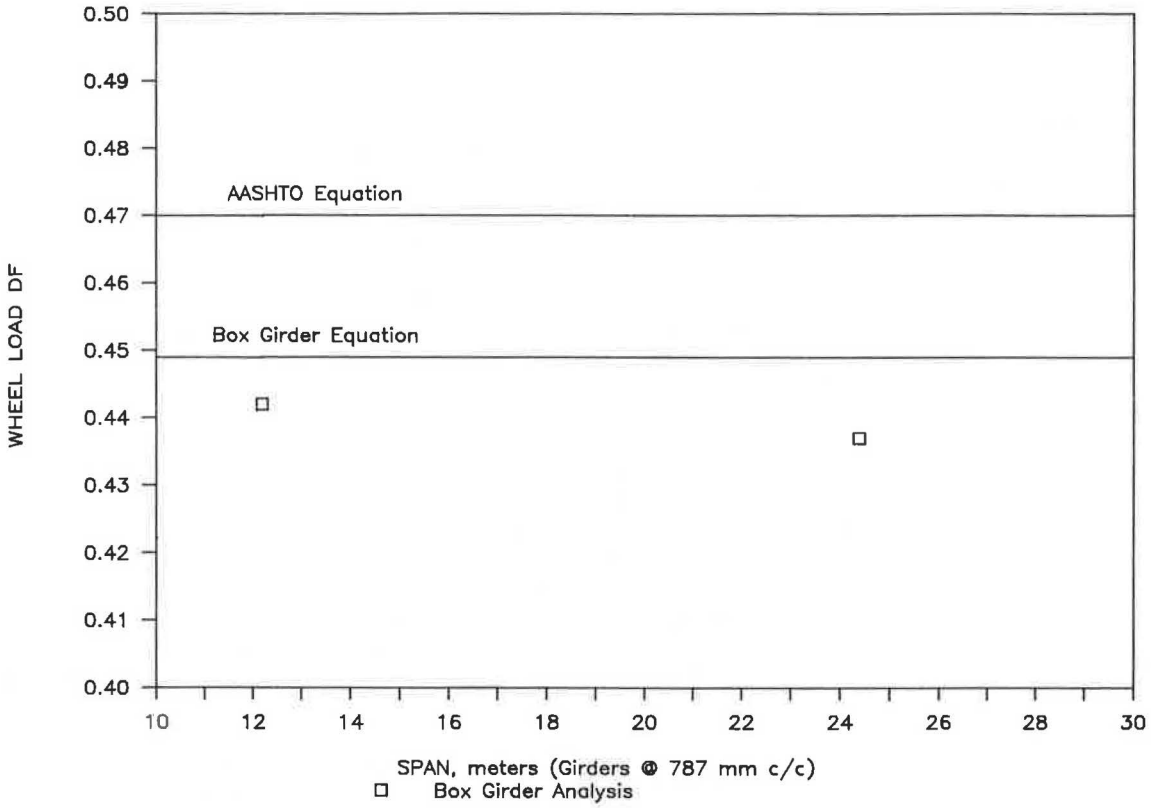
Two of the half-girders were joined with a prototype bolted splice design (Figure 7). Unlike a bolted splice for an I-girder, this splice offers a clear top flange for installing a bridge plank deck. The test splice was designed to provide bending and shear capacities of 90 and 75 percent, respectively, of the box net section capacities. The other two half-girders were joined by field welding procedures using stick electrodes (Figure 2, joint detail).

A finite-element analysis model of the welded splice girder was developed before testing to predict deflections and stresses for both the torsional and flexural loading tests. The ANSYS code was again used to develop the model (Figure 8) from plate elements.

TORSIONAL TESTS

For torsional loading, the test girders were set up on a structural testing floor, as shown in Figure 9. The girders were braced by crossframes fabricated for hot-rolled steel angles. A torsional loading frame (Figure 10) using two opposing-action hydraulic jacks was designed and fabricated to fit tightly around each girder. Dial gauges accurate to 0.025 mm were mounted at the end crossframes and on the load frame to measure horizontal deflections from which rotations were calculated.

Four bracing cases were tested. Case 1 had only the end crossframes in place. The midspan crossframe was added for Case 2. Because girder end warping distortions were observed in these two cases, internal plate diaphragms were welded to the bearing stiffeners at each girder end for Case 3. The midspan crossframe was removed for Case 4. Each case was



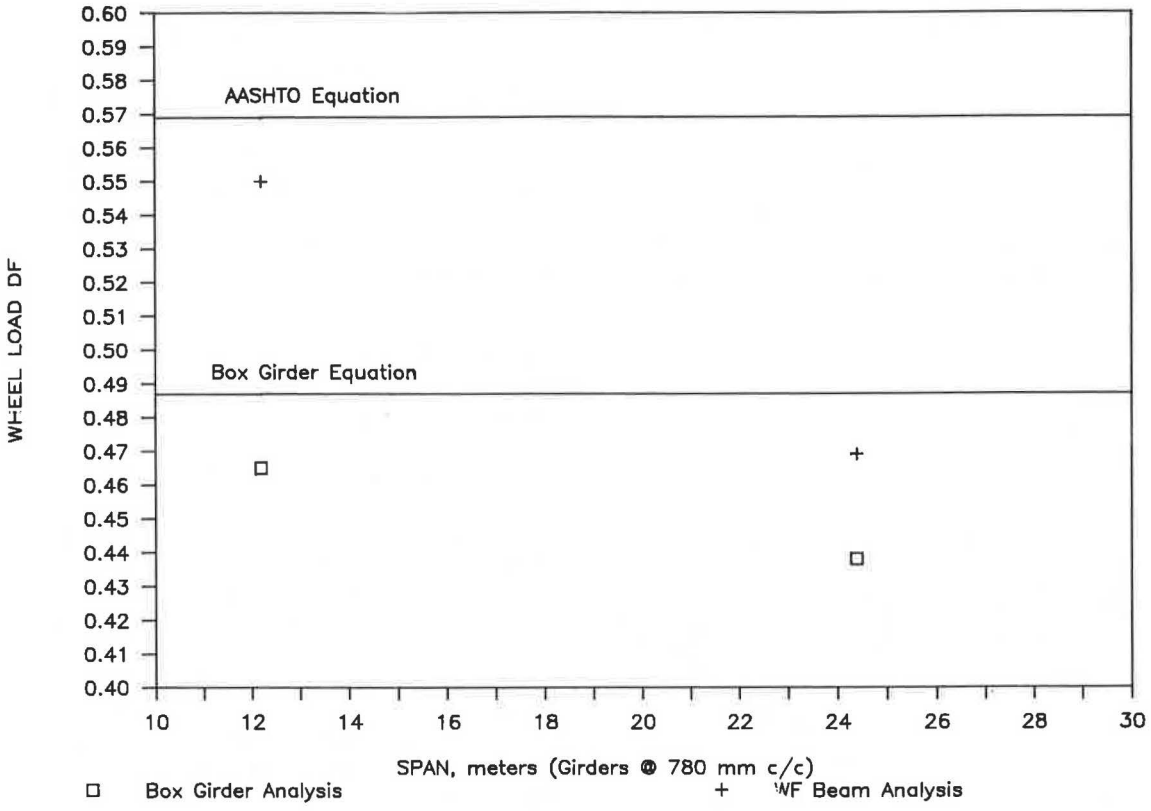


FIGURE 6 Wheel load distribution factors: (*top*) single lane and (*bottom*) two or more lanes.

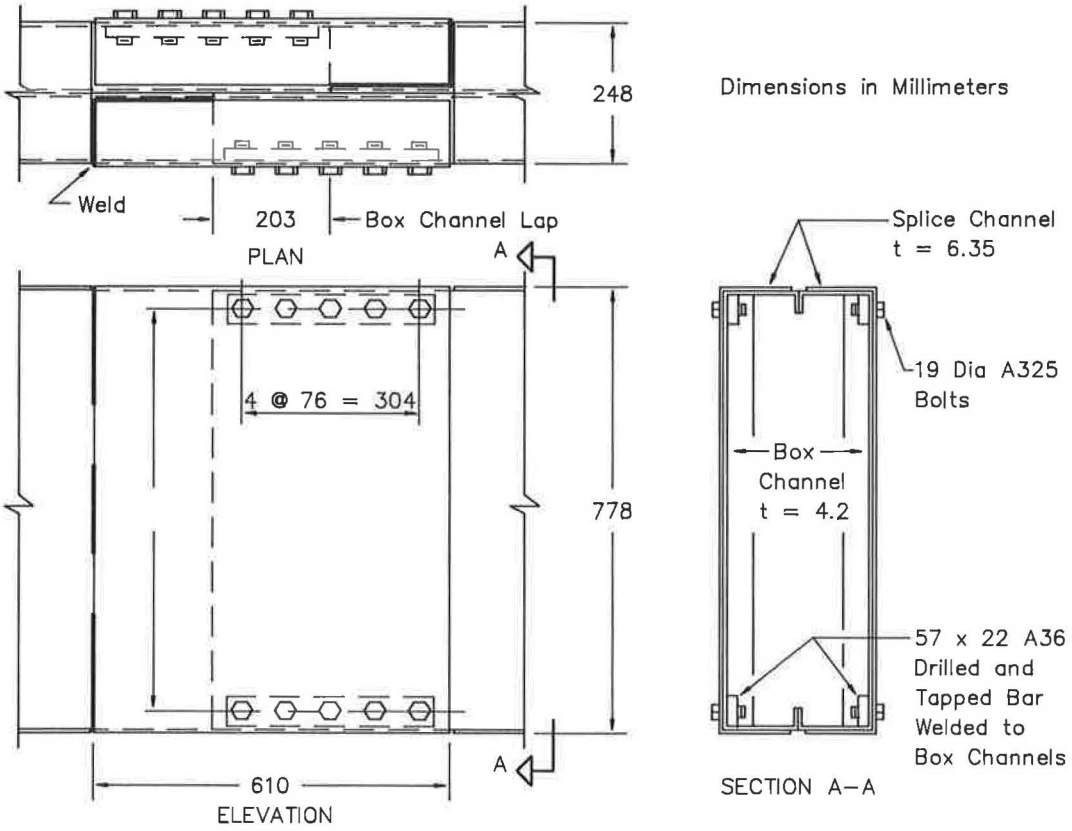


FIGURE 7 Prototype bolted splice.

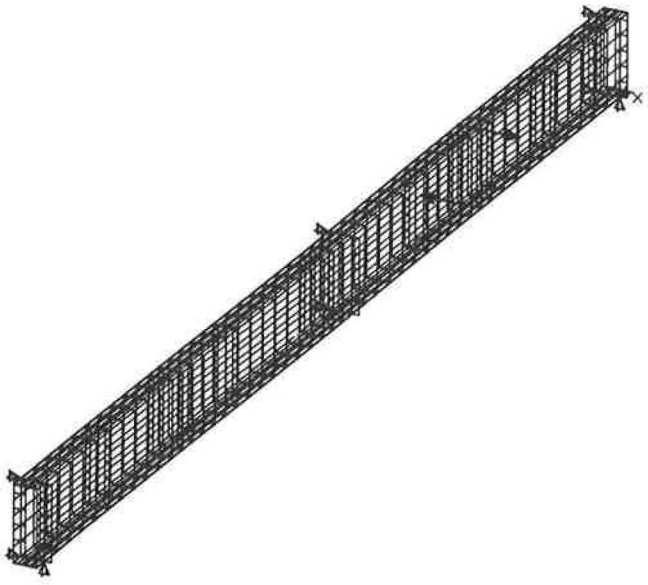


FIGURE 8 Test girder finite-element model. Torsional loading at north quarter-point end and midspan braces.

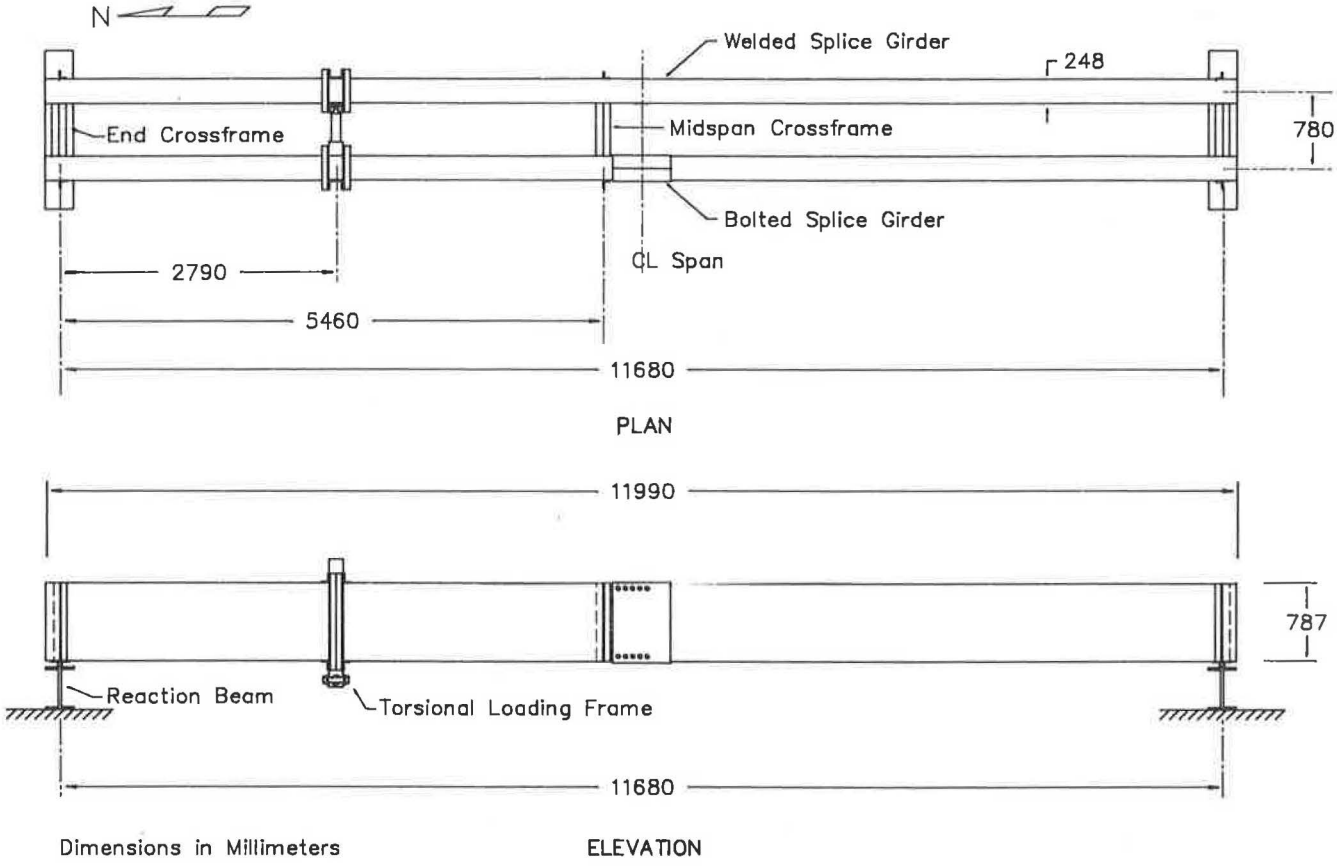


FIGURE 9 Torsional test setup.

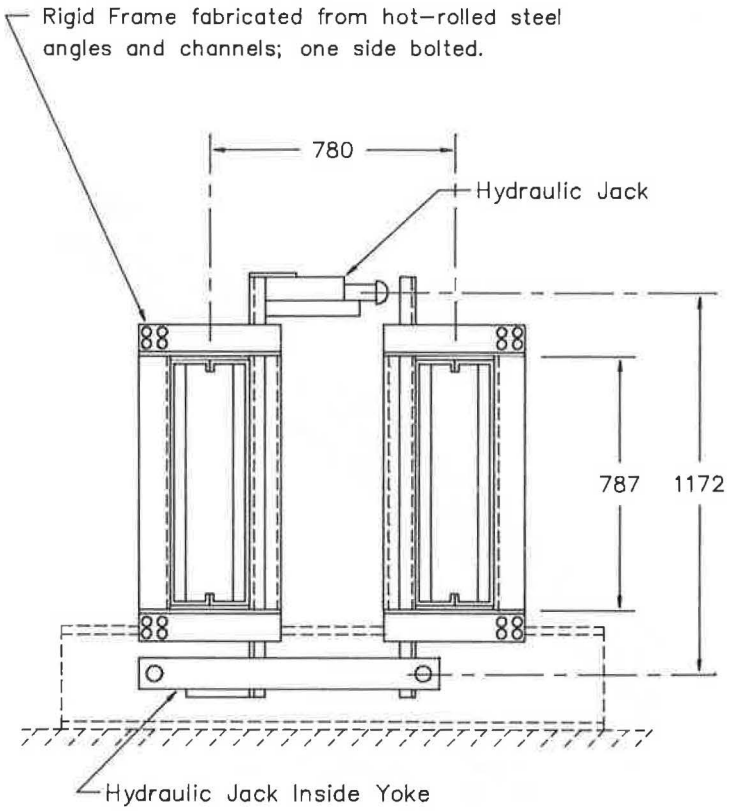


FIGURE 10 Torsional loading frame, sectional view (dimensions in millimeters).

loaded to approximately the maximum resisting torsional moment observed in the DF analysis models.

Net rotation at the land point is plotted as a function of the applied torsional moment (torque) in Figure 11. The internal diaphragms were effective in reducing warping, as can be seen by comparing Case 1 with Case 4 and Case 2 with Case 3 data in Figure 11. The diaphragms are used in all production box girders.

The average effective St. Venant torsional constants for the girders were backcalculated as 62 and 58 percent of theoretical for Cases 3 and 4, respectively. The 58 percent level was used in the distribution factor study to derive the box girder DF equations.

FLEXURAL TESTS

For flexural loading, the girders were set up individually on the structural testing floor (Figure 12). Loads were applied equally at the one-third span points using an MTS closed-loop, two-channel hydraulic test system (Figure 13). The test girder compression flange was braced laterally at the load points by knife-edge channel braces bolted to the load frames. Linear electrical transducers were used to measure vertical deflections to the nearest 0.025 mm at the loadpoints and at midspan. Loads were held constant while the transducer readings were taken.

Midspan deflection versus the ratio of applied load to ultimate load is plotted for both test girders and compared with the ANSYS model predictions, as shown in Figure 14. Flexural stiffness calculated from these plots was 95 and 87 percent of the ANSYS prediction for the welded and bolted splice girders, respectively. Bolt slip accounted for the lower stiffness of the bearing-type bolted-splice design. A reduction factor is applied to the effective box section moment of inertia for deflection calculations when bolted splices are used in production bridges.

The ultimate moment capacity of both test girders was controlled by the box section compression cover plate buckling strength. The actual yield point of the bolted splice channel steel was higher than anticipated, so the splice moment capacity was actually greater than that of the box section. Ultimate moment capacity was reached at 1.76 and 1.77 times actual allowable moment capacity for the welded and bolted splice girders, respectively, with the compression cover plate continuously welded along its longitudinal edges. Because this factor of safety level was slightly under the AASHTO nominal of 1.82, design criteria restricting (a) the cover plate maximum width-to-thickness ratio, (b) the spacing of intermittent cover plate welds, and (c) the maximum depth-to-thickness ratio of the webs were implemented to ensure a minimum factor of safety of 1.82 for production girders.

PERFORMANCE OF GALVANIZED STEEL IN BRIDGES

Although galvanizing has been in commercial use for over 200 years, it has been specified for entire bridge structures in North America only since the early 1960s. The first U.S. bridge with every structural member, fastener, and other steel components hot-dip galvanized was the Stearns Bayou bridge in Ottawa County, Michigan, constructed in 1966 (*6*). Since that time, several hundred all-galvanized bridges have been built in the United States.

The Stearns Bayou bridge was inspected in 1986 (*7*). It is in a rural atmosphere, is about 2 m above fresh water for most of its 128-m length, and is subject to winter salting. After 20 years of service, the beams and diaphragms were free of rust or stain. Coating thicknesses ranged from 76 to 140 μm (112 μm average). The bearing pads exhibited stain on areas subject to standing water and road deicing salts but still had coating thicknesses averaging 89 μm.

A monitoring program (*8*) comparing side-by-side galvanized and painted bridges in a suburban environment with deicing salt exposure has been in progress since 1970 in Indiana. Zinc-rich paint was used on some areas of the galvanized bridge. The last report (1987) indicated that both the galvanized and zinc-rich paint coatings were in good condition after 17 years of service. Minimum galvanized coating thickness was 94 μm. White rust (zinc oxidation) was showing on diaphragms adjacent to expansion joints and on bearing components. The painted bridge was recoated after 14 years and at 17 years demonstrated corrosion on diaphragms adjacent to expansion joints and on bearing components.

Atmospheric exposure test data on zinc and galvanized steel panels have been published for a number of locations in the United States, United Kingdom, and the Panama Canal Zone (*9*). These data indicate that the weight loss of zinc plates is linear with time and also correlates well with the weight loss of galvanized panels. Thus the performance of a hot-dip galvanized bridge at a particular site may be predicted by exposing weighed zinc or galvanized steel plates at the site for 2 to 3 years.

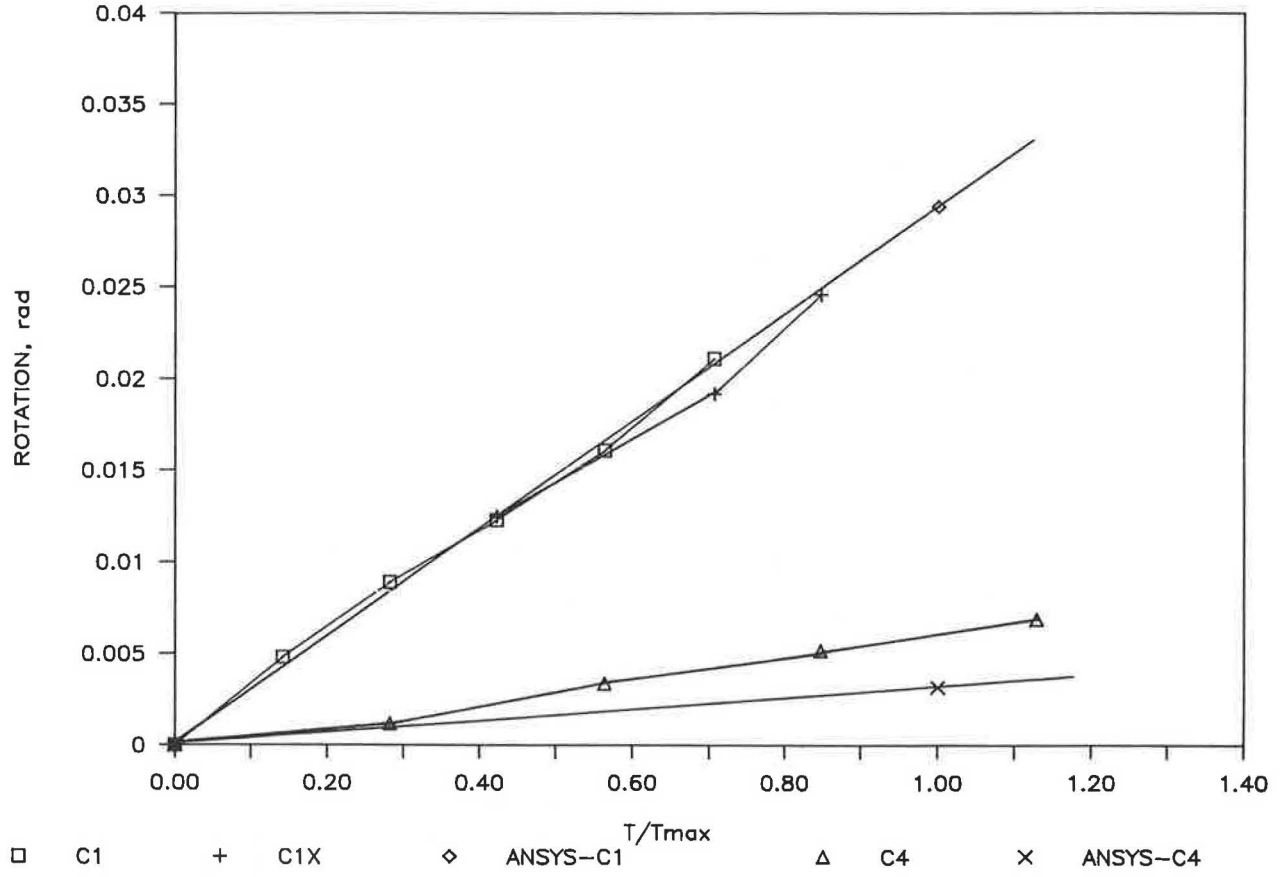
The following is a partial list of published estimates of years of coating life (years to rusting) for an 86-μm galvanized coating thickness (*9*).

Location	*Environment*	*Zinc Corrosion Rate (μm/yr)*	*Coating Life (yr)*
Miraflores, Panama Canal Zone	Mild marine	1.135	76.1
Limon Bay, Panama Canal Zone	Marine	2.654	32.5
Daytona Beach, Fla.	Marine	1.996	43.3
Brazos River, Tex.	Industrial marine	1.839	47.0
Middletown, Ohio	Semi-industrial	1.224	70.5
Phoenix, Ariz.	Rural desert	0.295	293

Galvanized coating provides both barrier and cathodic protection to the steel. The zinc provides sacrificial (cathodic) protection to the base metal when it is exposed by scratching or gouging. An experiment that demonstrated this was reported by Coburn (*9*). A galvanized steel plate was scored to the base metal with varying line widths and exposed to an industrial environment for 56 months. Visual inspection revealed no red rust in exposed widths up to 1.6 mm. Some surface rusting was demonstrated at an exposed width of 12.7 mm.

LIFE CYCLE COSTING OF GALVANIZED BRIDGES

Life cycle costs should be considered when a coating is chosen for steel bridges. The cost of galvanizing must be related to field performance to determine life cycle costs.



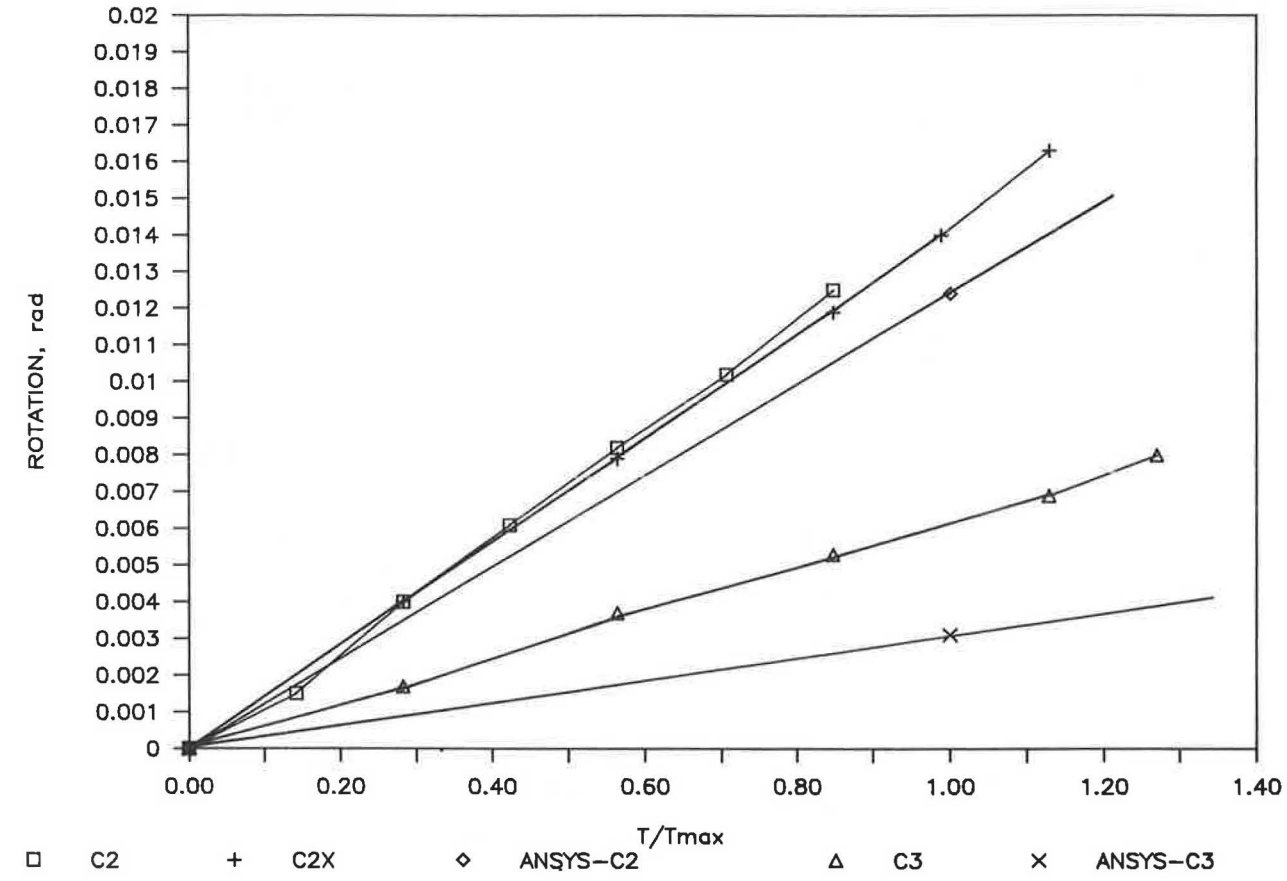


FIGURE 11 Torsional test results: (*top*) end bracing only and (*bottom*) end and midspan bracing.

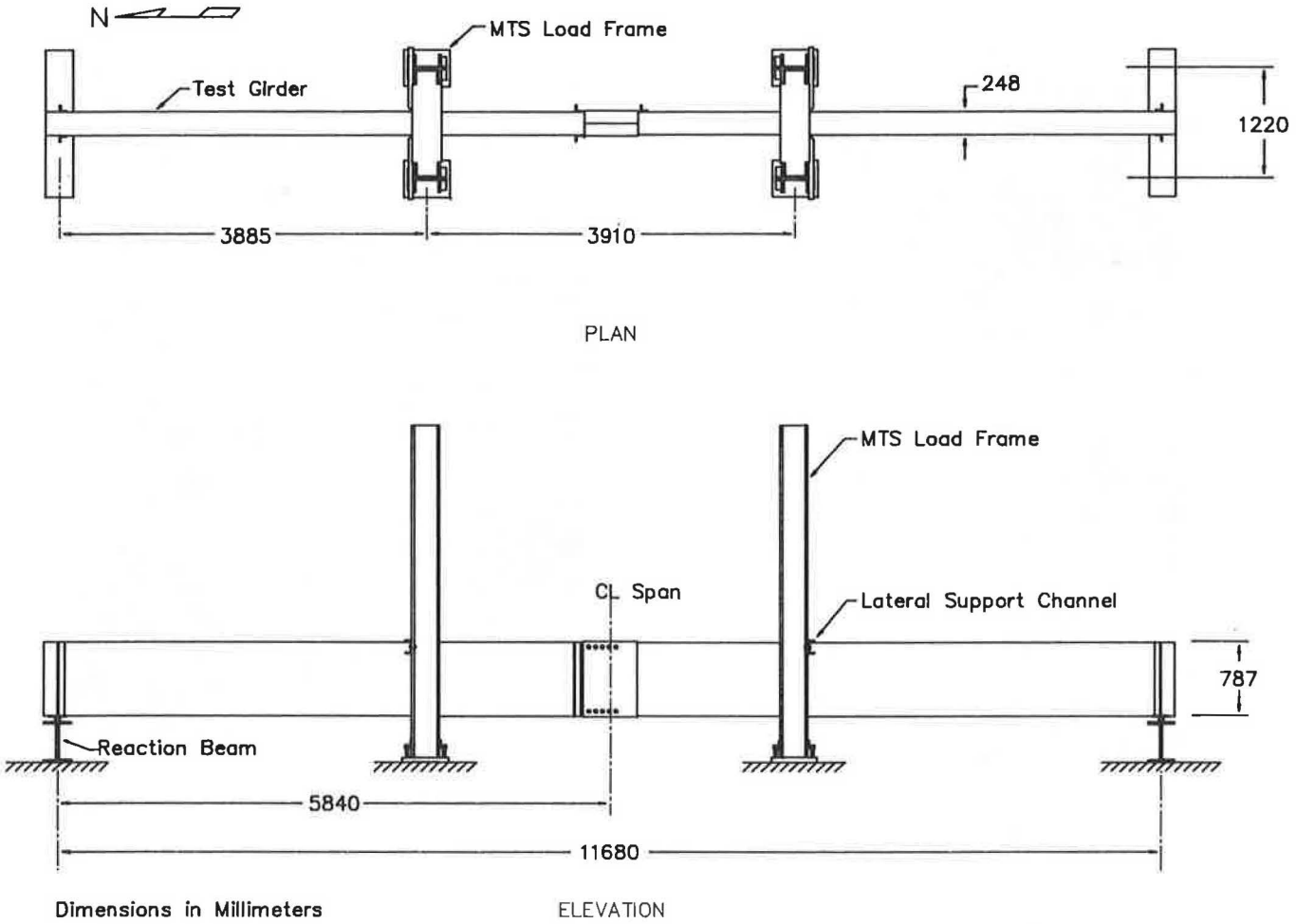


FIGURE 12 Flexural test setup.

A detailed discussion of life cycle costing is provided by Coburn (*9*).

The results of a study on the Stearns Bayou bridge show that when the bridge was built in 1966, the total galvanizing cost was $8,750 (*6*). The painting estimate was $8,600 including blast and shop cleaning plus field painting. Assuming a liberal 20-year service life for the paint and a 5 percent annual inflation rate, the cost of painting was estimated at $17,000 after the first 20 years, $46,000 after 40 years, and $322,000 after 80 years. The galvanized coating life was estimated at about 80 years (*7*). The additional $150 galvanizing cost resulted in a 27 percent annual return on investment by postponing recoating costs.

CONCLUSIONS

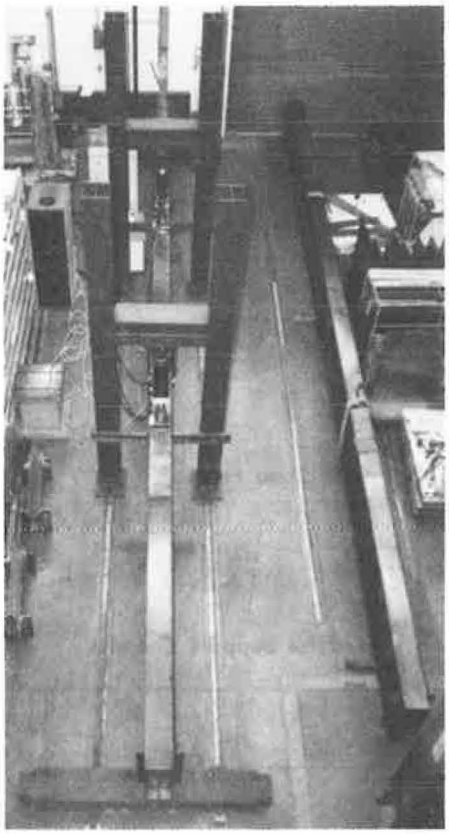
Typical installations, design practices, wheel load distribution factors, structural component tests, and galvanized steel performance data have been described for a new galvanized cold-formed steel bridge system. A number of these short-span bridges have been designed and built recently on low-volume roads in Ecuador.

Key structural components of the system are the cold-formed steel box sections used as main longitudinal girders. These provide greater redundancy and were shown by finite-element analyses to provide better wheel load distribution than conventional wide-flange beam or I-girders. Full-scale structural tests confirmed that strength and stiffness of the box girders can be accurately predicted using state-of-the-art cold-formed steel design technology with some restrictions on geometry and welds.

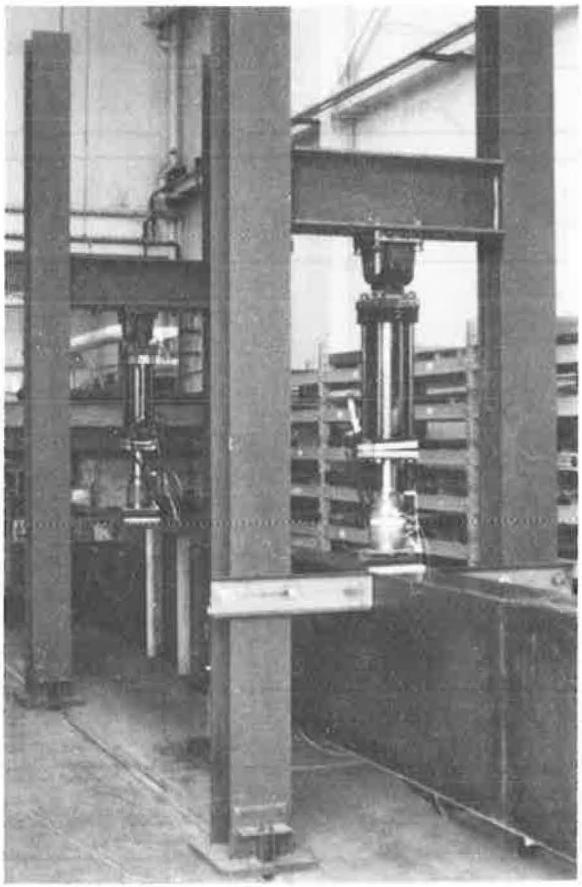
Durability performance data and life cycle costing analyses on galvanized steel bridges in the United States indicate that the Ecuadorian bridge design offers a durable, maintenance-free, and economical system when site environmental conditions are suitable for galvanized coatings.

ACKNOWLEDGMENTS

The authors wish to thank the management of Armco Inc. and its Latin American Division for sponsoring this work. Special thanks are due W. E. Brown, J. N. Macadam, and K. L. Norris of Armco Research and Technology for their valuable contributions to the testing program, and to J. Hidalgo



(a)



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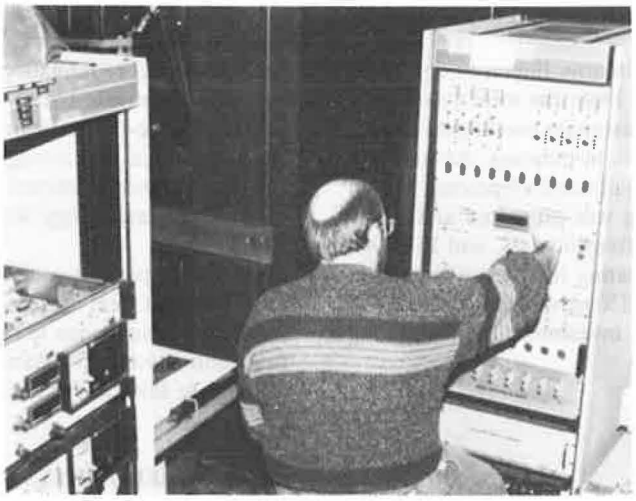

(b)

FIGURE 13 Flexural test equipment: (a) view from above, looking south; (b) left—load control unit, right—deflection read-out system; and (c) load reaction frames, actuators (jacks), and lateral braces (looking north).

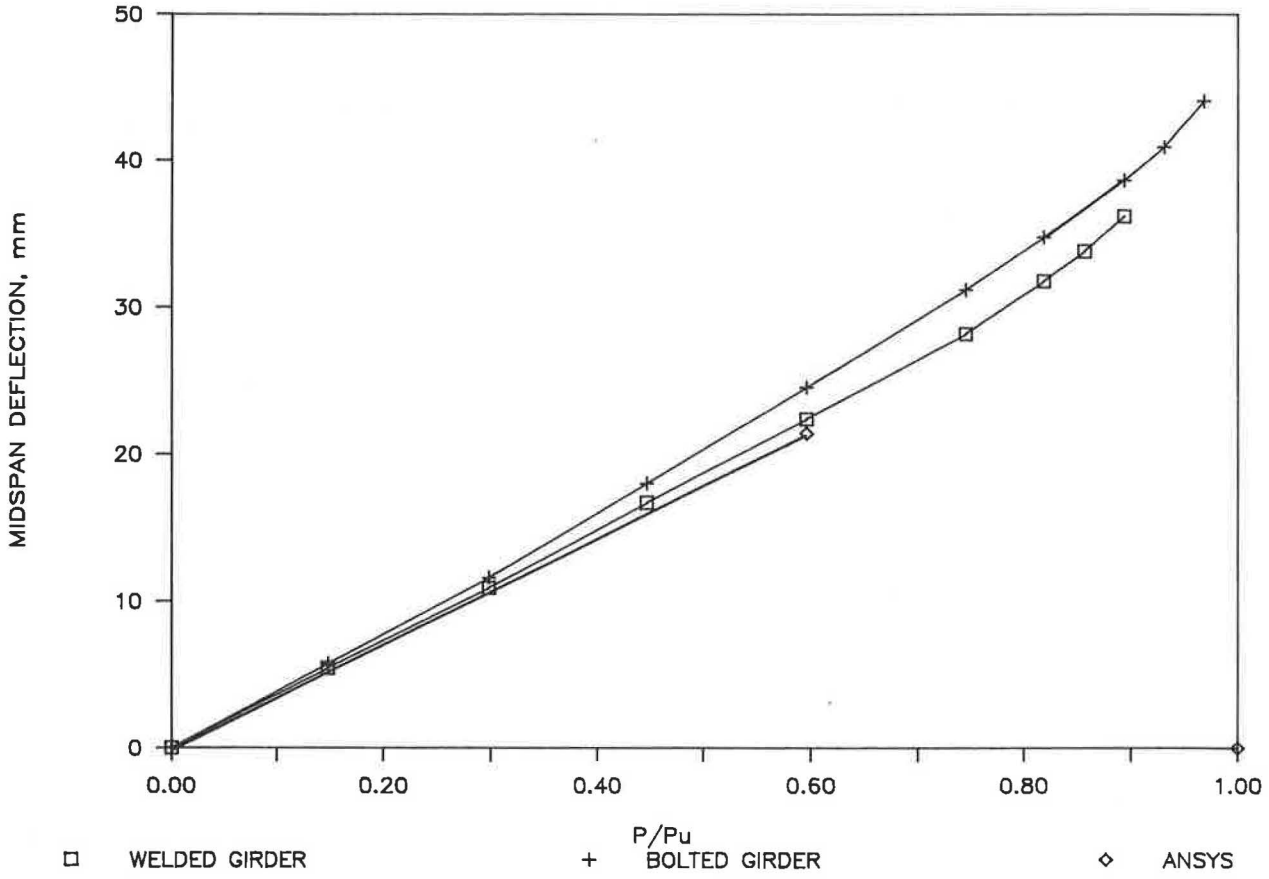

FIGURE 14 Flexural test results for prototype cold-formed box girders compared with ANSYS model predictions.

and A. Silva of Productos Metalicos Armco S. A., Quito, Ecuador, who provided valuable insights and support.

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New Folded-Plate Bridge Culvert System

MAHER K. TADROS, AMIN ALEINEA, AND AHMAD M. ABDEL-KARIM

A new precast concrete bridge/culvert system is described. The system is particularly suitable for a roadway crossing over a waterway, where the span is relatively short (20 to 60 ft), and the clearance between the high water level and roadway surface is greater than 6 ft. The system can be built quickly by most contractors, making it an attractive alternative for replacement of many of the nation's deficient secondary road bridges and culverts. It consists of precast concrete folded-plate elements supported on ledges of end walls, which are in turn supported on drilled piers. Variations of this basic system are possible to suit other span ranges and crossing geometries. The system is extremely efficient as its components use available space to increase structural cross section depth and to reduce dead loads that would otherwise be heavy. Other benefits are offered by this new system, including short construction time, economy, and ease of construction. The new system is compared with an existing system; a cost analysis demonstrates potential savings as high as 20 percent over the traditional system. Plans are under way to construct a full-scale functional prototype in eastern Nebraska.

The annual reports to Congress on "Highway Bridge Replacement and Rehabilitation Program," by the FHWA have repeatedly voiced concern about the state of the nation's bridge infrastructure. The Ninth Annual Report included data about 577,710 highway bridges in the National Bridge Inventory (NBI). The FHWA indicated in their report that as of June 30, 1988, 23.5 percent of those bridges were classified as structurally deficient and 17.7 percent as functionally obsolete. There is need for economical systems to rebuild the rapidly deteriorating bridges. Where spans are relatively short and vertical clearances allow, culverts are often used as economical alternates to bridges. Among the most commonly used culvert systems are low-profile corrugated metal culverts, precast concrete box culverts and arches, and cast-in-place (CIP) concrete box culverts. However, a new and promising concept of an economical and efficient bridge/culvert system is currently being developed at the University of Nebraska. The final design of the new system will be completed in the near future, followed by the construction of a full-scale functional prototype structure. Several projects have been identified as possible test sites in the eastern Nebraska region.

The new concept uses precast folded-plate elements as the primary load-carrying components. Folded plates are thin and efficient structural members that used to be popular in the 1960s for building roofs. The large structural depth of folded plates allows them to span long distances at lower material cost compared to other structural systems. Popularity of reinforced-concrete folded plates for building roof framing has diminished in recent years because of the high cost of CIP concrete formwork, the distinctive appearance of folded plates (which is not anymore fashionable to some), and because of possible snow load buildup. None of these problems are valid for the proposed usage of folded plates. Precasting significantly reduces formwork cost and improves quality. Appearance is not a problem as the folded plates are covered with soil. Finally, a folded-plate system, as will be shown later, carries a significantly smaller total soil load than an alternate CIP reinforced-concrete box.

Several other advantages are offered by the proposed system over the currently used culverts or bridges for the same crossing characteristics.

1. A folded-plate system would require a shorter overpass span than the corresponding bridge (or narrower width than the corresponding culvert barrel length).
2. When a waterway is being crossed, the new system does not require any waterway channel alterations or waterway rerouting during construction.
3. Keeping the waterway unchanged would provide the least interruption to wildlife habitat.
4. In comparison with CIP construction, the construction of the proposed system is much quicker, resulting in less traffic disruption and public inconvenience.

The proposed system is described here for roadway-waterway crossings with the following characteristics.

1. Width of the highest water surface of 20 to 60 ft.
2. Width of bridge, that is width of roadway including shoulders, up to 50 ft.
3. Clearance of 6 to 10 ft between the highest water level and surface of roadway.

These characteristics apply to a large number of crossings on low-volume roads and secondary highways. Where intermediate supports are allowed, use of the proposed system can be extended to include a variety of applications and dimensions beyond these limits.

A brief description of the present competing bridge/culvert systems is presented accompanied by a comparison with the proposed system. A detailed description of the proposed system along with conceptual plans and sections are given. Functional and economical comparisons are made between a preliminarily designed folded-plate bridge and a recently constructed box culvert in Sarpy County, Nebraska. It is shown that along with several advantages, construction cost savings as high as 20 percent could be achieved by using the proposed system instead of the existing box culvert. And finally, several design aspects of the proposed bridge system are considered.

PRESENT SYSTEMS

There are several bridge/culvert systems available on the market; each lends itself to certain sites with different character-

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istics. For the purpose of comparison, available competitive systems are briefly discussed below. Only those systems that can be used for sites with the characteristics mentioned will be discussed.

Single or Multiple Tubular Culverts

One or more pipes are generally used across the roadway to allow water flow under the roadway. The number of pipes and their diameters depend on the volume of water flow expected in the waterway channel. A large volume of fill is usually required for this type of construction. These systems generally require modification of the geometry of the waterway channel, and they are suitable for sites with low volume of flow.

CIP or Precast Concrete Culverts and Bridges

Concrete box and arch culverts are rather popular. CIP construction costs may be relatively high because of the high cost of forming. On the other hand, precast concrete box culverts are relatively small in size and limited by the availability of precasters and heavy equipment contractors. Conventional CIP and precast/prestressed concrete bridges are usually used for wide crossing sites and for highway overpasses. They are most economical for relatively low vertical clearances that do not allow for soil cover under the roadway.

Low-Profile Corrugated Metal Culverts

Steel as well as aluminum culverts are available. Typically, the culvert is assembled by bolting a number of transverse corrugated plates together to form the box section. The box can be assembled in place or preassembled on site, then lifted into place. During erection and backfilling, the assembled box is flexible and sensitive to distortion. Transverse stiffeners are sometimes used to preserve the shape of the box during erection and backfilling. Strict backfilling sequence is necessary for proper installation of this type of structure. These culverts have relatively short service life (as short as 20 years). They require rerouting of the waterway during construction and a relatively large volume of fill and site work.

Prestressed Concrete Patented Systems

Two patented culvert systems are available: Con/Span and Hy-Span culverts. Although these culvert systems are different in shape from each other, they are similar in nature and material. Both systems are composed of three-sided precast concrete segments (inverted U shape) joined together in the direction of the waterway to form a single-cell culvert. Multiple cells can be formed by putting units next to each other. Both of those systems are designed to bear on spread footings or bottom slabs. The construction of such structures offers no savings over traditional boxes in the volumes of excavation and backfill, rerouting of the waterway during construction, or interruption of the natural channel and wildlife habitat.

DESCRIPTION OF THE PROPOSED SYSTEM

The proposed folded-plate bridge/culvert system is composed of four major components. The components from the ground up are the foundation, the end walls, the wing walls, and the folded plates (Figures 1 and 2).

Foundation

The foundation of this bridge system consists of four CIP concrete drilled piers or precast concrete driven pile groups. (Only four piers or pile groups are required, as will be explained later.) Other types of foundation are possible depending on the span of the bridge/culvert, the topographical character-

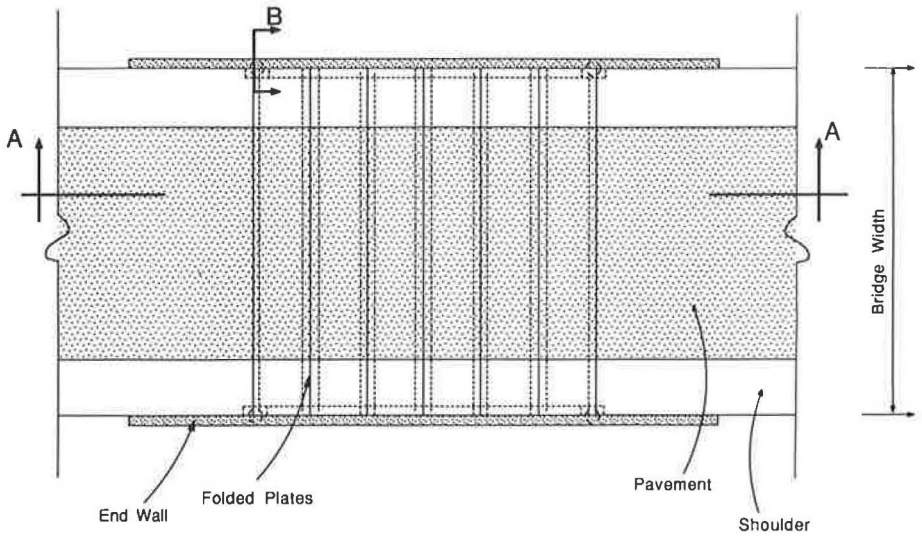

FIGURE 1 Folded-plate bridge plan view.

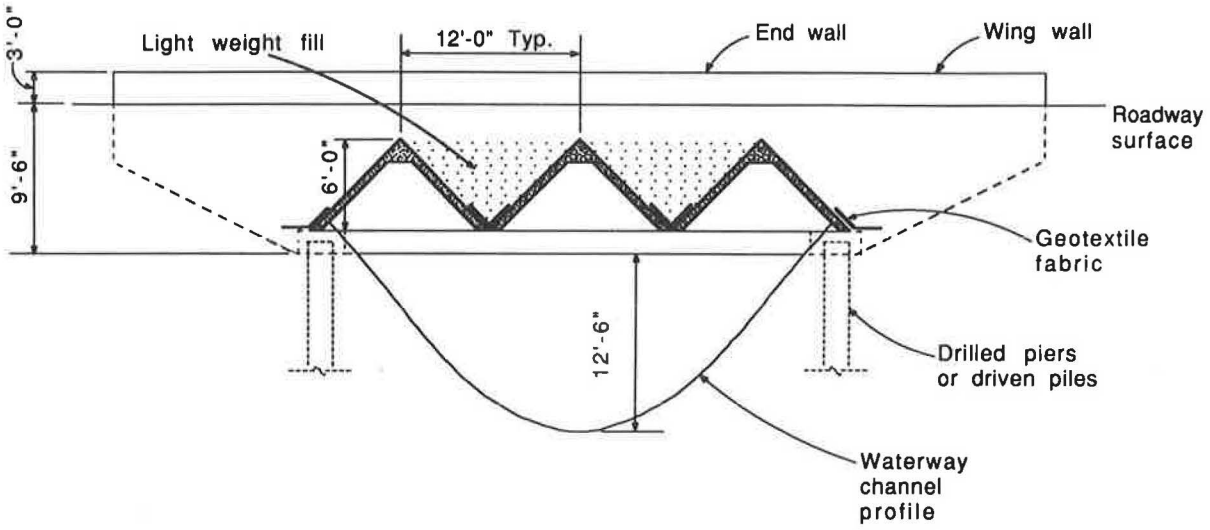

FIGURE 2 Folded-plate bridge, Section A-A.

istics of the site, the design loads, and the subsoil conditions. Drilled piers are particularly attractive for the proposed system because they occupy small space and can be installed, by a relatively large number of contractors, without disturbing the natural profile of the channel.

End Walls

The end walls serve three primary functions: first, each end wall spans across the waterway between two piers or pile caps and supports one end of the folded-plate units. Second, they retain the road embankment. Third, they can be made high enough to work as a traffic barrier.

Wing Walls

The wing walls in the proposed system are continuations of the end walls that cantilever on either side of each end wall. They retain the road embankment within the distance required to achieve smooth grade transitions.

Folded Plates

The folded-plate units span between the end walls, across the roadway, to transfer dead and live loads to the end walls (Figure 1). In addition, they stabilize the end walls in the out-of-plane direction because the end walls as well as the wing walls are subject to lateral earth pressure. Ninety-degree folds are assumed in the preliminary design and are the most economical for forming purpose; other configurations are possible depending on the crossing dimensions and loading. The advantages of using reinforced-concrete folded plates as the main structural members as well as the advantages of the entire system compared to a CIP box culvert system are discussed in the next section.

COMPARISON WITH AN EXISTING SYSTEM

In order to best demonstrate the efficiency and advantages of the proposed system, a comparison between an existing CIP reinforced-concrete twin-box culvert built in Sarpy County, Nebraska, and a preliminarily designed folded-plate alternative for the same site is presented. The construction of the box culvert was completed in April of 1988. Plan view and cross sections of the box culvert are shown in Figures 3 and 4, respectively.

Six bids were submitted for the construction of the box culvert. The bids ranged from $105,920 to $191,980 with an average of $142,170. It was felt that, for comparison purposes, the average bid price most accurately represents the actual box-culvert construction cost. The cost estimate of the proposed folded-plate bridge was carried out conservatively by local precast concrete suppliers and contractors to closely represent its actual construction cost. The estimated total construction cost was $109,800, which is 20 percent less than the average bids for the box culvert. The primary reasons behind the lower construction cost of the new system versus that of the box culvert in comparison are discussed in the following paragraphs.

Less Volume of Excavation and Backfilling

In order to construct the box culvert, the contractor had to excavate to a level slightly lower than the bed of the waterway channel, and the excavation had to extend a few feet outside the design dimensions of the culvert on four sides. After completely curing the CIP concrete culvert, a large volume of fill had to be hauled in, placed, and compacted on all sides and on top of the culvert to build the roadway embankment above and to smooth the grades around it. Considerably less volume of excavation and backfilling is required for the construction of the proposed bridge system. Only a small amount of fill needs to be placed above the folded-plate units to build the roadway embankment. Also, because of the geometry of the

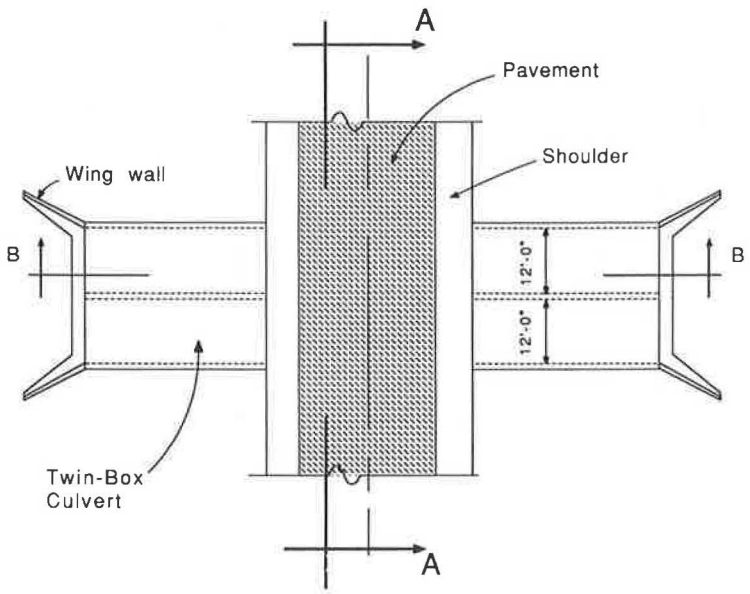

FIGURE 3 Twin-box culvert plan view.

folded plates themselves, less fill is required to bring the roadway embankment to the final pavement elevation (Figure 2).

Less Site Work

Because of the nature of the proposed bridge system and its geometry, little site work is required for its construction. This is mainly because of the function of the end walls as retaining elements, which eliminates the need for gradually sloping the road embankment on both sides of the road. Unlike the construction of the culvert, which requires a great deal of grading to smooth the surfaces of the new fill, the proposed system has a level, well-defined fill geometry that requires only minimal grading work.

Less Materials

In order to accommodate the slopes of the roadway embankment, the box culvert had to be 100 ft long. The folded-plate bridge, on the other hand, is only about 38 ft wide. This is mainly because of the fact that the length of culvert is parallel to the waterway, whereas the width of the folded-plate bridge is parallel to the waterway. The total concrete volume needed for the CIP concrete culvert was approximately 350 yd^3, and the total concrete volume needed for the CIP and precast concrete members in the proposed bridge system was conservatively estimated to be 100 yd^3. The box culvert needed 250 percent more concrete than the proposed bridge did for the same crossing.

Shorter Construction Time

Most of the folded-plate bridge members can be precast concrete members including the pile foundations. This means shorter construction time and less susceptible-to-weather construction process than that of the CIP concrete box culvert. Shorter construction time means more economical construction, shorter traffic disruption time, and less public inconvenience.

No Interference With the Natural Waterway Channel

When used for sites with characteristics similar to those mentioned previously, construction of the proposed bridge system causes no interference with the natural profile of the waterway channel. This is true because the foundations of the proposed system are the only portions extending below the water surface and they occur outside the perimeter of the running water. Hence, unlike the CIP box culvert, the new system does not require an extensive hydraulic study or rerouting of the waterway, nor does it cause interference with the wildlife habitat.

Less Overall Cost

Less volume of excavation and backfilling, less site work, less materials, and shorter construction time add up to less construction cost. That was reflected clearly in the difference between the construction cost of the box culvert in comparison and that of the proposed folded-plate system. As much as 20 percent cost saving can be accomplished by using the proposed system versus the CIP concrete box culvert.

DESIGN CONSIDERATIONS OF THE PROPOSED SYSTEM

Because the detailed analysis and design of the proposed bridge system are still in the early stages, little technical information and mainly the concept of the proposed system are provided.

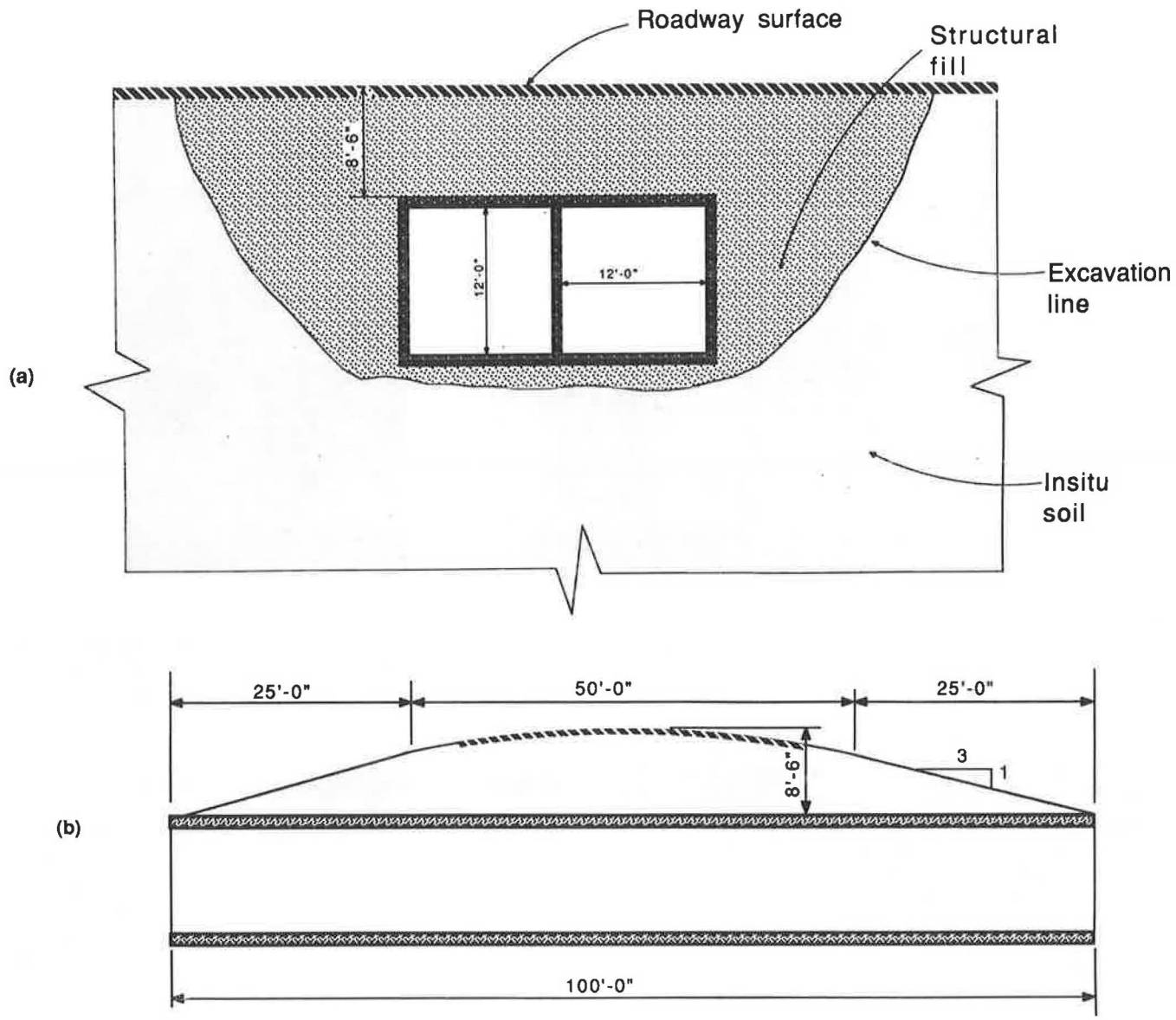

FIGURE 4 Twin-box culvert, (a) Section A-A, (b) Section B-B.

The two major components of the bridge system, the end walls and the folded-plate units, will be considered precast concrete members hereafter. CIP concrete construction of these two components will preclude the purposes behind developing this system—namely, economy, short construction time, and minimum interference with the waterway channel. So far, the structural design of the proposed bridge/culvert system is still in its preliminary stages. The more extensive study of the structural design and detailing will take place in the near future. However, a few of the most important design considerations are discussed in the following paragraphs.

Design of Folded Plates

The folded plates span between the end walls to support their own weight, the weight of road embankment and pavement, and the design live loads caused by traffic. At the same time, the folded plates have to support the end walls in the out-of-plane direction as the end walls are subject to lateral earth pressure from the fill and road embankment. Design live loads will depend on the classification of the overpass as specified by the *AASHTO Standard Specifications for Highway Bridge*. The distribution of soil pressure resulting from the road embankment and design live loads, however, is difficult to predict. The unusual geometry of the top surface of the folded plates results in a complex soil-structure interaction. The problem is further complicated by the material nonlinearities of soil and concrete, as well as the additional nonlinearity caused by cracking of concrete. Boundary nonlinearities also exist at the interface between the soil and structure. In order to model these effects as accurately as possible, a special-purpose finite element program is required. One such program, Culvert Analysis and Design (CANDE) will be used

in the analysis phase of this investigation. CANDE is capable of accounting for all of the nonlinearities, and outputs such information as the soil pressure and internal forces (shear and bending moment) at the nodal points of the folded plates.

One of the interesting design concerns is how to connect the adjacent folded-plate units to each other and how to connect them to the end walls. The connections shall transfer the loads from the folded plates to the end walls, and vice versa, with the least local stress concentrations.

The precast folded-plate units can be either conventional or prestressed concrete members. The need for prestressing will be determined by the thickness of the embankment, design live loads, and the span of the plates.

Design of End Walls

The main function of end walls is to support the folded plates and to transfer the loads to the foundations. But they also retain the road embankment, and if desired, they can be made deep enough to project above the road embankment to work as a traffic barrier or guard. For that reason, the folded plates will have to support the end walls in the out-of-plane direction. Therefore, a connection cable of transferring forces in the out-of-plane direction has to be designed between the folded plates and the end walls (Figure 5). When the bridge span is short, erection of the end walls is simple as they are short and relatively light. For relatively long-span bridges, however, the end walls become too heavy, and erecting them becomes a challenge that will have to be investigated in the final design stages. For roadway widths greater than those specified above, more than one folded-plate span can be used across the roadway. In these instances, intermediate walls as well as intermediate foundations have to be used (Figure 6).

The effect of temperature changes will be investigated and accounted for when designing the end walls. The connections of end wall to foundation will require special attention because they have to transfer the largest, and most important, forces in the system (Figures 5 and 6).

Foundation Design

The most appropriate foundation for this system is CIP-drilled piers. They can carry a considerably large axial load in most soil conditions, they have relatively small areas, and they can be easily installed by most contractors. Friction piers, bearing piers, or friction-bearing-combination piers can be used depending on the foundation loads and the subsoil conditions. Driven-pile foundation is another candidate for the folded-plate bridge. Only four pile groups with CIP concrete pile caps are required. However, type, cross-sectional area, and length of piles will depend on the foundation loads and subsoils conditions. Regardless of the type of foundations used, they have to be designed and detailed to support vertical dead and live loads, lateral wind or seismic loads, and loads caused by temperature changes.

SUMMARY AND CONCLUSION

Economical systems are needed to rebuild the rapidly deteriorating bridges and culverts in the country. A few bridge and culvert systems are presently available on the market for waterway crossings with spans up to 60 ft and vertical clearances greater than 6 ft. A new promising bridge/culvert system is being developed at the University of Nebraska for sites with

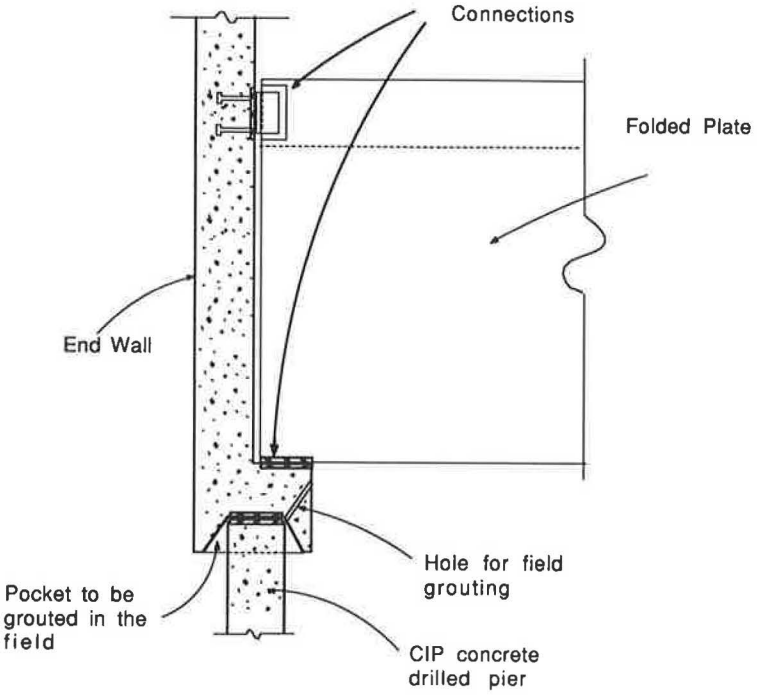

FIGURE 5 Folded-plate to end wall connection, Section B-B.

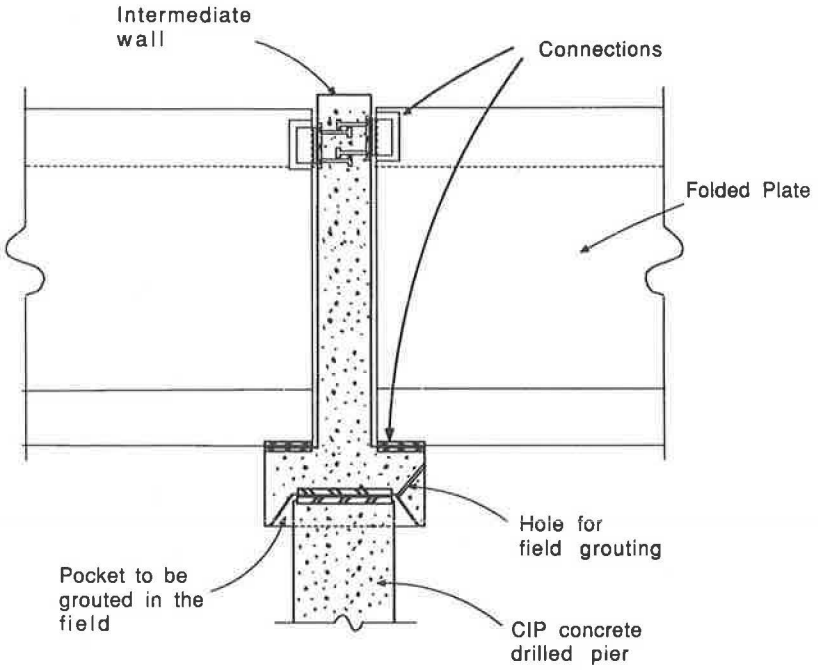

FIGURE 6 Typical folded-plate to intermediate wall connection.

these characteristics. The new system concept makes use of precast-concrete folded plates as the main structural elements of the bridge. In order to demonstrate the potential for cost savings, a construction cost estimate based on preliminary design of a folded-plate bridge was compared with the construction cost of an existing culvert system. The comparison indicated that the proposed system has obvious potential time saving and approximately 20 percent cost savings over the box culvert. In addition, the proposed bridge system permits no interference with the natural waterway profile. Although the system is being limited to certain dimensions and applications at this stage of its development, it can be further adapted for use in roadway overpasses and for long and wide bridges where intermediate supports are allowed. Research is underway at the University of Nebraska to further develop this new concept both from design and constructibility points of view.

Considerations for Fish Passage Through Culverts

FRANK E. VOTAPKA

Minimizing obstructed fish passage through culverts is an important step that an engineer must take when designing and installing culverts. This process will often require consultation with a fish biologist to ensure that all fish passage considerations are recognized. The existing research is reviewed and synthesized. A good set of recognized guidelines to be used to design culverts for roads in rural areas is provided.

Unhindered fish passage at stream crossings is an important consideration in the engineering of the extensive road network of the United States. The identification and planning for replacement of existing road drainage structures to facilitate fish passage is an area of high national need. This responsibility will require unprecedented cooperation between biologists, engineers, and hydrologists. The dollars associated with drainage structure replacement will be staggering, as is the potential impact to remaining fish runs.

A set of broad guidelines is outlined for the engineer and fish biologist concerning what is needed to design, construct, and maintain an acceptable structure with fish passage capabilities. Many of the principles for fish passage through culverts can be adapted to the design of any drainage structure. References of fish passage through road drainage structures are reviewed and synthesized. This synthesis does not detail the technical methods used for analysis and design; it will not replace the need for project level consultation between the fish biologist and the engineer; nor will it replace the need for professional evaluation and design on a site-specific basis. However, it should aid such professionals in their effort to obtain an acceptable structure.

Many stream crossings are culvert installations. These installations consist of a variety of road drainage structures, including corrugated metal pipes, box culverts, and natural bottom arches. Recognition has been increasing that these crossings should not only be engineered for road alignment and grade but also for allowing unhindered fish passage.

For many fish, migration is essential to the survival of the species. For example, fish that travel from the ocean up river to spawn (anadromous) begin a maturation process geared to culminate when they reach their spawning habitat. Improperly selected and placed culverts can be barriers to such migration runs creating the ultimate irony of denying fish access to their spawning areas after they have swum hundreds, if not thousands, of miles. However, anadromous fish are not the only fish that migrate.

Many resident fish species such as trout, pike, and grayling migrate upstream and downstream during their life cycle seeking a variety of aquatic habitats that might include spawning, rearing, or hiding habitat. Although these migrations may only be a few miles, they are just as important for the long-term survival of the species and maintenance of fish production.

Ensuring unobstructed fish passage to each stream is an important step that the engineer must make. Often it requires consultation with a fish biologist to ensure that all fish passage considerations are met. Poorly planned, designed, or constructed culverts may become serious problems to the production of fish runs and in some cases the survival of fish species. Today's engineer and fish biologist must not only consider culvert design for efficient water passage, but also heed such factors as fish species, water velocity, water depth, culvert length and slope, and specific streambed conditions.

Because there are often differing philosophies on what a proper structure should be and what acceptable impacts are, sufficient information will be provided to agency managers and technical experts on the preparation of standardized policies that have often been confusing in the past.

FISHERIES CONSIDERATIONS

Adult Fish

The vast majority of past research and reports regarding fish passage at road drainage structures has been oriented to adult anadromous fish. The most common approach at assessing fish capabilities has been to divide swimming speeds of adult fish into various activity categories (such as cruising, sustaining, and burst speed) (*1*,*2*). Typically, the sustained speed has been identified as the appropriate criterion for determining whether water velocity would block migrating fish. Table 1 presents some of the swimming capabilities of some common fish. Each species of fish has been found to have different swimming capabilities.

Juvenile Anadromous Fish

Although the majority of fish passage efforts have historically been geared to adult anadromous fish passage (especially salmon and steelhead trout), resident fish species and juvenile anadromous species also exhibit a variety of upstream migrations. Skeesick (*7*) was one of the first authors to document a consistent upstream migration of juvenile coho salmon in the fall of each year. The 10-year study on Spring Creek (Wilson River, Oregon) did not investigate the reasons for the upstream migration of juveniles, although it speculated that "the juve-

Kootenai National Forest, Libby, Mont. 59923.

TABLE 1 SWIMMING CAPABILITIES OF FISH SPECIES

Fish Species	Maximum Capability (ft/sec)	Acceptable Range (ft/sec)	Reference Source
Juvenile Salmon		0–4	Saltzman and Koski (*3*)
Trout and Steelhead		0–3	Metsker (*4*)
Adult Cutthroat		0–4	Saltzman and Koski (*3*)
Trout and age 1+ Steelhead		0–3	Metsker (*4*)
Adult Sea-run	6.4–13.5[1]	0–8	Saltzman and Koski (*3*)
Cutthroat Trout	11.4[2]		
Adult Coho	12.2–17.5[1]	3.4–10.6	Bell (*1*)
Salmon	10.6–21.5[2]	0–8	Saltzman and Koski (*3*)
		0–8	Lauman (*5*)
Adult Chinook	14.5–22.1[1]	3.4–10.8	Bell (*1*)
Salmon	10.8–22.4[2]	0–8	Saltzman and Koski (*3*)
		0–8	Lauman (*5*)
Adult Steelhead	12.0–26.8[1]	4.6–13.7	Bell (*1*)
Trout	13.7–26.8[2]	0–8	Saltzman and Koski (*3*)
		0–8	Lauman (*5*)

NOTES: 1 From Calhoun (*6*)
2 From Bell (*1*) using Trout.

nile coho moved into the small streams to escape the high flow, turbid-water environment in the main rivers in the spring."

Under some conditions, upstream migrations of juvenile anadromous fish and movements into tributaries do occur. These migrations are very much at risk by drainage structures in general including structures that are designed with only adult fish migration in mind. In a stream system managed for wild fish production, blocking juvenile fish movements into tributary streams can have an impact on fish production in the watershed.

Regulations that require fish passage capabilities for juvenile fish have been slow in developing. One exception to this has been in Washington State, which has site-specific requirements (as determined by the regional habitat manager) to provide upstream salmonid fingerling passage to overwintering habitat.

Resident Fish Species

Resident fish species also exhibit a variety of in-stream movements. These include adfluvial spawning migrations of cutthroat trout, and other salmonid fish species, as well as in-stream movements of resident fish from unknown causes. Like juvenile anadromous fish, upstream movements of resident fish are also commonly blocked by culverts and drainage structures. Water velocities that can accommodate adult salmon and steelhead passage commonly obstruct resident fish species. Culvert outfalls easily jumped by older resident fish can block younger fish.

In streams containing only resident populations of fish, the decision is regularly made (consciously or inadvertently) to obstruct upstream fish passage. Because resident fish species can generally perpetuate their populations above natural (and presumably man-caused) barriers, fish production is commonly assumed to be relatively unchanged in year-round stream systems, (Genetic segregation, however, could characterize the upstream fish populations.) If a road drainage barrier were placed below an occasionally dry channel, though, a complete loss of resident fish production above the barrier would ultimately follow.

In some streams, fish blockage has been purposely caused by installing culverts and highway structures to obstruct fish movement. This practice has occurred in a number of locations, particularly in the eastern United States to prevent the movement of undesirable fish species. In blocking the passage of undesirable fish species, the passage of nontarget species is also typically obstructed.

Impacts of Delayed Fish Migration

A predominant philosophy that has historically governed fish passage considerations has been that fish migrations should not be delayed at road drainage structures. This belief, while being theoretically attractive, has conflicted with the reality that most drainage structures impede fish passage to some degree. In addition, many fish species impose limitations on their own upstream migrations during periods of heavy runoff, or during adverse fish passage conditions. In some instances, the attempt to avoid any interference with fish passage has led to the placement of large drainage structures that are extremely expensive, and probably impede the passage of fish at lower stream flows.

Although many culvert installations have caused delays in fish migrations, there has been remarkably little research on the effects of various delay lengths. The majority of research has been directed at assessing the impacts of delayed migration on Arctic grayling and a few other species (*8–10*). The most definitive study on the effects of spawning delays on Arctic grayling is by Carlson (*11*). That study demonstrated that some delay did not appear to adversely affect spawning effort or success. However, as delays lengthened an increasing adverse impact to spawning occurred.

Control of Sediment

Release of sediment into a stream can have some serious impacts on fish and fish habitat. There are several obvious ways that sediment can affect fish populations.

1. Sediment can settle on spawning beds. It will eventually settle into the voids of the gravel and either smother the eggs or newly hatched fish (alviens) by hindering subsurface water circulation. It can also create cobble embeddedness, which effectively seals the spawning gravels to potential spawning fish.
2. Sediment can clog or abrade a fish's gills causing suffocation or infection.
3. Sediment will reduce the visibility in the stream, hindering the fish's ability to seek food.
4. Sediment also smothers and displaces the invertebrate organisms that serve as a food source for fish.

The most effective way to control sediment at a culvert site is to dewater the site, install the culvert in the dry, and ensure that the site is as stable as possible before diverting the stream back through the pipe. In many cases, it is possible to take advantage of low-flow conditions and realign the stream slightly to allow the structure to be installed. At low flow, fish are usually not migrating. Working in the dry is usually the least-cost method of construction for the contractor; it also eliminates many of the water quality problems with downstream water users. There are many methods available to the contractor for sediment control during construction. Some basic methods are documented by Dane (*2*). Additionally, more sophisticated methods of sediment control such as use of geotextiles have evolved as the issue of sediment control has become more involved.

ENGINEERING CONSIDERATIONS

There are some general guidelines to consider when installing culverts in fish streams. Figure 1 (*12*) shows some of the undesirable conditions for fish passage through culverts. The end result can be an installation that is unsuitable for fish passage. With proper design and construction considerations followed, a crossing can usually be suitable for fish passage.

1. In those places where fish passage could be a problem, bottomless arches, bridges, or pipe arch culverts partially buried are preferred over round pipes. This is especially true if pipes are over 100 ft long, if threatened or endangered species are involved, or if the stream gradient is over 2 percent.
2. There are two opinions on multiple culvert installations. The first opinion is that one large culvert is preferred over several smaller ones, because the larger one is less likely to plug with debris and will carry water at a lower velocity (*12*). The second opinion is that multiple installations are all right. In fact, if there are multiple installations, one pipe can be sized to pass peak flows and the second could be designed to pass fish. However, occasionally adult fish have been observed to jump into culverts with the higher velocity in multiple installations. Because of such behavior, even though fish passage may be provided at one culvert, the fish may not choose that culvert to pass.
3. Water velocities in smooth-bottom culverts are usually two to three times those in corrugated metal pipes when all the slope and pipe size parameters are equal. Migrating fish use the corrugations along the pipe as resting areas as they migrate through the pipe. Larger corrugations (> 6 in.) are preferred over smaller ones (*10*).
4. Culvert diameters must be adequate to pass the maximum expected design flows. Excess sedimentation from washed-out culverts and the associated road fills can damage spawning and resting habitat. Most agencies have their own design parameters, but recommended minimum design parameters for those agencies that have not established such parameters should be to design a culvert to pass a 50-year flood at a static head and a 100-year flood with a headwater depth.
5. Culverts should be designed and installed to keep the velocity of the water passing through the pipe equal to the predicted stream velocity at design flows. Many authors advocate maximum grades for culverts (see inlet drops), but these can result in producing supercritical flows at the culvert inlet which can themselves become barriers to fish passage.
6. Two major considerations in designing culverts should be the maximum acceptable water velocity and the minimum acceptable water depth [usually not less than 6 in. (15.24 cm) for resident trout and at least 12 in. (30.48 cm) for adult anadromous fish]. Velocities for design flows through a structure should not exceed the natural stream velocity of a 10-year floor (Q10).
7. Migrating fish can usually tolerate some delay in migration. An acceptable practice is not to require flow conditions that are suitable for fish passage during the 5 percent period of the year when flow peaks are their highest (*12*). Fish do not generally migrate at highest peak flows, so this practice should cause little disruption of normal fish migration. This practice also often results in substantial savings in construction cost for fish passage.
8. As a general practice, culvert baffles are not recommended in lieu of installing a larger pipe or using a reduced pipe gradient. However, at times baffles cannot be avoided. If such baffles are installed, they must be designed such that the culvert can still handle peak flows and the baffles themselves do not become debris accumulators.
9. Often, culverts are needed to be installed in streams with high gradients. In these cases, effort should be made to provide for resting pools and bank protection for several hundred fish above and below the culvert installation. In order to prevent scour and stream degradation, it is important to maintain stream stability.
10. More design consideration should be given to the location of the stream crossings. Too often, the roadway alignment dictates the culvert location. At least equal consideration should be given to hydraulic criteria. Where fish passage is important, structures should be placed where the stream bed is the straightest to ensure that natural meanders are not cut off. This will result in higher stream velocities.
11. When used on the upstream end of culverts, riprap should be placed carefully. Dislodged riprap can result in reduced hydraulic capacity for culverts and in supercritical flows that hinder fish passage. Table 2 can be used by the designer to assist in sizing the riprap.
12. The use of concrete aprons at culvert openings is not recommended. The aprons make the fish passage difficult or

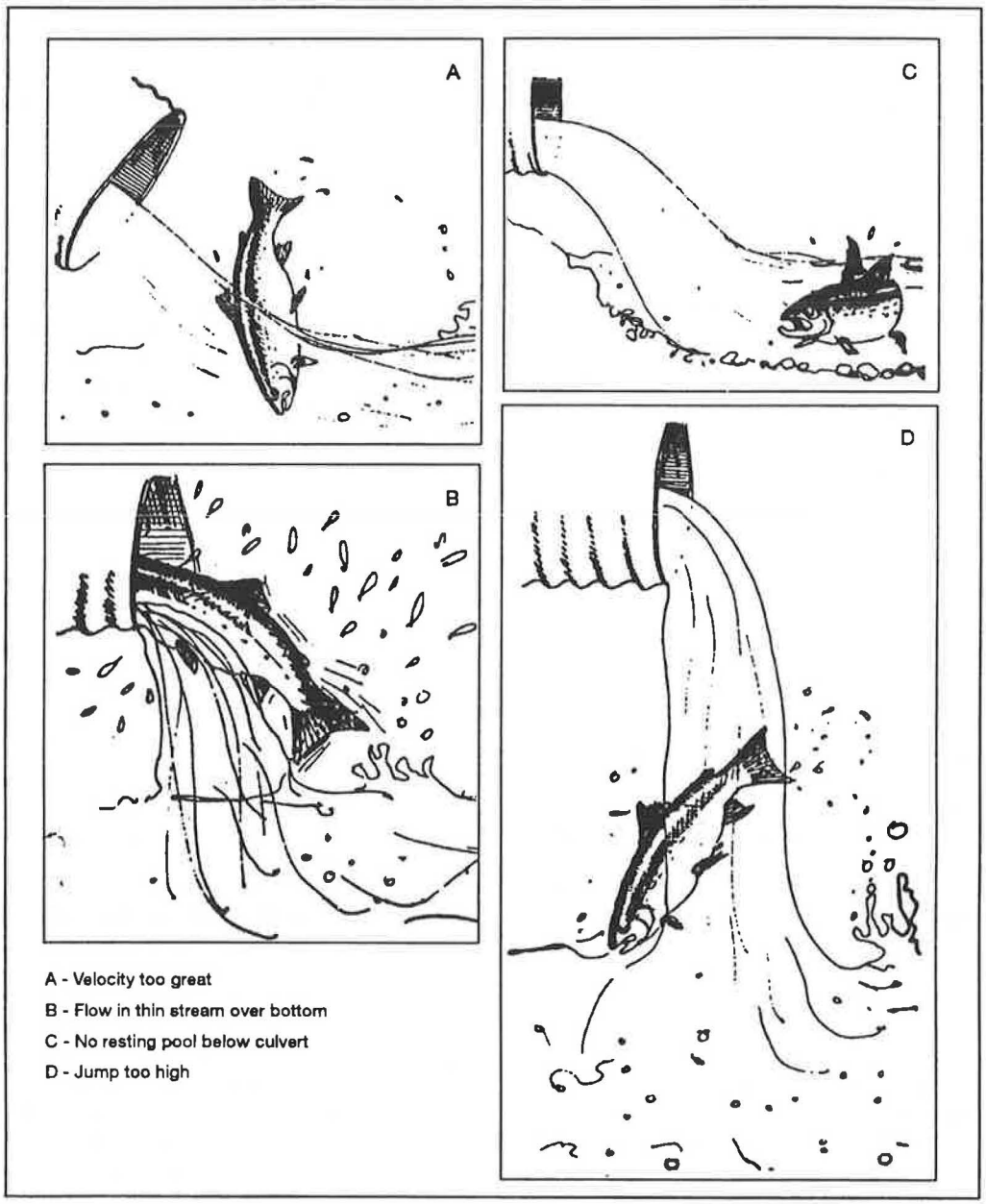

FIGURE 1 Common conditions that block fish passage (*11*).

impossible because of increased velocities resulting from lower roughness coefficients and from the fact that many aprons are installed at steeper gradients than the culvert.

13. An outlet pool with tailwater control capabilities should be designed and constructed at the downstream end of a culvert with critical migratory fish problems. The length and width of the outlet pool should be twice the diameter of the culvert ($2D$), and the bottom elevation of the pool should be at least 2 ft below the invert elevation of the culvert outlet.

14. All rehabilitative work within the stream channel should be completed before the stream is diverted back through the completed culvert. Disturbance of the natural habitat should be minimized when armoring of the stream is needed. For example, when the road embankment needs to be protected, the embankment itself should be armored instead of the stream banks above and below the culvert.

HYDRAULIC CONSIDERATIONS

In order to properly design any culvert, the engineer will need to know the design parameters including fish species, age, its maximum water velocity requirements, and the length the fish can swim without a resting area so that a culvert can be sized and designed properly. Baffles, natural rock boulders, or other resting areas may need to be designed to properly pass fish by providing resting areas.

TABLE 2 RECOMMENDED RIPRAP SIZE (BASED ON CORPS OF ENGINEERS PUBLICATION EM-1110-2-1101, pp. 11–31)

Water Velocity (fps)	Percent of Riprap Smaller Than						Manning's "N" Value	AASHTO Riprap Gradation Class (a)	Forest Service Riprap Gradation Class (b)
	100		50		10				
	wt. (lbs) 1	size (feet)	wt. (lbs)	size (feet)	wt. (lbs)	size (feet)			
6.5	25	.67	5	.40	1	.23	0.034	-	I
7.5	50	.88	10	.50	2	.27	0.036	-	II
8.0	100	1.04	25	.67	2	.27	0.0375	-	III
9.5	150	1.17	75	.95	10	.50	0.039	Facing	IV
9.75	250	1.42	100	1.04	15	.58	0.040	-	V
10.0	400	1.67	175	1.23	20	.62	0.041	Light	VI
10.5	700	2.02	300	1.52	25	.67	0.042	-	VII
11.5	1600	2.67	600	1.92	50	.88	0.044	1/4 Ton	VIII

1.) Assuming A Specific Gravity of 2.65

Riprap Size Classes based on:

a.) Standard Specifications for Highway Bridges, AASHTO, 14th Edition, 1989

b.) Forest Service Standard Specifications for Construction of Roads and Bridges, April 1985

Water Velocities in Culverts

The swimming abilities of fish depend on the fish species. In addition, the size of the fish also has a bearing on the ability of the fish to negotiate the stream. Small fish, including juveniles and adult trout, are much more susceptible to velocity barriers. A relationship exists between the size (length) of the fish and their swimming capability—the smaller the fish, the lower the velocity tolerance. Therefore, if fish passage at a culvert crossing is geared only for adult fish, anadromous and residential fish production would be impacted upstream from the culvert.

In order to aid road designers in estimating water velocities through culverts, the USDA Forest Service has produced a series of culvert velocity curves based on Manning's equation (*12*). In addition, there are several computer software programs available that can calculate velocity, depth of flow, and other parameters.

Culvert Outfall Barriers (Perching)

Culverts can often have insurmountable barriers to migrating fish when the outlet of the culvert is so far above the tailwater that fish cannot enter the pipe. This condition generally occurs when the outlet is installed above the stream elevation or velocities through the pipe are high enough to wash out the stream below and cause a cutting downstream. This condition is referred to as an outfall barrier or culvert perching.

A perching problem can be corrected by installing one or a series of low head dams below the culvert outfall. Often these dams may be nothing more than hand-placed rock, gabion baskets filled with local rock, concrete sills, or logs. The purpose of these dams is to raise the tailwater elevation and flood the culvert outlet. The end result is enhanced fish access and reduced culvert velocity at the outlet. The low-head dams downstream should therefore be limited to 1 ft in height or have a weir to allow for fish passage. It may be necessary to install several downstream dams to get the desired elevation if the culvert outfall barrier is excessive.

The general purpose of these tailwater control structures is four-fold:

1. The structure provides a resting pool for migrating fish before they swim into the higher velocity culvert.
2. Creating a backwater into the pipe allows for adequate water depths in the culvert.
3. A backwater reduces the velocity at the culvert outlet, thereby enhancing fish migration.
4. Much of the energy from the culvert is dissipated in the pool created by the tailwater control section. The pool provides a transition zone between the culvert and the natural channel downstream.

Determining if a perching problem will occur is essential in proper culvert design. One method for calculating the probability is to use Manning's equation to determine the flow in the pipe. Once the flow is determined, the velocity can be

ascertained by dividing the flow by the area. If the velocity is 10 ft/sec or larger, perching at the culvert outfall will likely occur. This can be mitigated by providing tailwater structures as outlined or by riprapping the outlet.

Inlet Drops

On about 10 percent of the culverts examined in detail in Alaska, Kane and Wellen (*13*) observed drops at the culvert inlet. These drops can become a barrier to upstream fish migration at high or even moderate flows. In all cases, they felt that these drops were caused by deposition of material from either the natural streambed or adjacent road way embankments. When the deposition was from natural streambed material, it was from lower velocities at the upstream end of the culvert because the culvert was laid on a flatter grade than the stream. Deposition from adjacent roadway embankment occurred when riprap material rolled down the embankment to rest in front of the culvert inlet.

In either case, the deposition resulted in supercritical flow conditions creating hydraulic jumps within the culvert. Even at moderate flows, fish passage can be significantly reduced.

In order to control velocity in culverts, many designers have advocated that a maximum gradient be used (see Table 3). In many cases, this procedure results in the culvert's being installed at less than the stream gradient, particularly in mountainous streams. It will result in lower velocities only if the culvert is enlarged to handle the same design discharge. However, such designs may result in reduced sediment-carrying capacity and in debris being deposited at the upstream end of the culvert. Significant deposition can create the inlet drops discussed previously if the designer does not consider the effects on sediment-carrying capacity. In addition, the outlet end of the culvert needs to be installed at or below the streambed or a culvert perching problem can occur.

Should a designer want further information on this subject, it is recommended that he consult several documents. One such publication is *Design of Depressed Inlet Culverts* by Jordan and Carlson (*17*).

Culvert Alignment

Problems with fish passage can occur if culverts are not aligned with the natural stream channel. Sudden changes in stream flow direction can result in turbulence and erosion of the stream bank and roadway excavation. When stream meanders are substantially cut off with culvert installations, the slope of the culvert and the resulting stream velocities will be greater than the original stream velocities. Such conditions are conducive to scour of the culvert outlet.

In order to prevent scour or erosion at a culvert site, the following points should be considered:

1. Avoid locating a culvert crossing at or near bends in the stream. The channel should be as straight as possible.
2. Avoid aligning the culvert so that culvert flows are directed into a stream bank. If a road crossing is not perpendicular to the stream, the culvert installation should be skewed.

TABLE 3 SUGGESTED MAXIMUM GRADIENTS

Publication	Suggested Maximum Gradient
Evans and Johnston (*12*)	At or near zero
USDA-Forest Service (*14*) R5	3% less than stream grade
State of Alaska, DOT&PF, Hydraulic manual	Flat grade
Morsel et al. (*15*)	0.5%
Dane (*2*)	Less than 5% with baffles
Dryden & Stein (*8*)	Prefer 0% gradient; less than 5% with baffles
Gebhards and Fischer (*16*)	Less than 0.5%

3. Construct a culvert outfall basin to dissipate the energy of the water flow, which many times is concentrated at the culvert outlet. The length and width of such a basin should be about twice the diameter of the culvert and the depth should be about 2 ft (0.61 m) below the invert elevation of the culvert outlet.
4. The stream channel and the streambed below the culvert should be stable to avoid lowering natural control points in the stream and subsequent streambed lowering either above or below the installation. These possibilities can be prevented by using many ideas suggested previously.
5. Cuts, fills, and other disturbed areas should be appropriately armored during construction. Armoring can be accomplished by planting grass and brush where water will constantly attack the disturbed areas. Consideration of armoring should be given where improper culvert alignment cannot be avoided and banks are threatened by the direction of flow.
6. Undermining of culvert inlets or outlets can be prevented by construction of cutoff walls attached to the bottom of the culvert and extending perpendicular to the streambed. Use of aprons at the inlet or outlet should be avoided.

SUMMARY

The success of fish migration through culverts depends on the swimming ability of the fish and the hydraulic conditions of the culvert. Properly designed and constructed culverts can help minimize the impact on fish passage.

Because culverts are more economical than bridges, it is appropriate to evaluate when to use culverts and to predict the effects of such culvert installations. There are enough documented successes in the use of culverts that to eliminate them entirely in a design because of fish passage is unreasonable. The aspects of culvert design and operation relative to the existing information have been examined.

Ideally, a culvert installation should not change the conditions that existed prior to that installation. This means that the cross-sectional area should not be restricted by the culvert, the slope should not change, and the roughness coefficients should remain the same. Any change in the conditions results in velocity changes and changes in sediment transportation capacity of the stream.

A truly successful culvert design would include matching the velocities of the fish's swimming zone in the culvert to

the swimming capacity of the design fish. Unfortunately, not enough research has been done to make this an acceptable portion of a culvert design. This approach is preferred because velocities in the occupied zone can be more readily reduced by increasing boundary roughness than can the mean velocity of the entire culvert, which is presently the accepted method in culvert design.

Relatively simple guidelines, when combined with the expertise of an experienced fish biologist and an engineer, can be used to reduce the problems resulting from installation of culverts in the streams containing migrating fish.

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Fifth International Conference on Low-Volume Roads 1991

Papers presented at the Fifth International Conference on Low-Volume Roads May 19–23, 1991 Raleigh, North Carolina

A peer-reviewed publication of the Transportation Research Board

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. 1991

Transportation Research Record 1291
Price: $84.00

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Printed in the United States of America

Library of Congress Cataloging-in-Publication Data
National Research Council. Transportation Research Board.

International Conference on Low-volume Roads (5th : 1991 : Raleigh, N.C.)
Fifth International Conference on Low-volume Roads : papers presented at the Fifth International Conference on Low-Volume Roads, May 19–23, 1991, Raleigh, North Carolina.
p. cm.—(Transportation research record, ISSN 0361-1981 ; no. 1291)
ISBN 0-309-05071-5
1. Low-volume roads. I. National Research Council (U.S.). Transportation Research Board. II. Title. III. Title: 5th International Conference on Low-volume Roads. IV. Series: Transportation research record ; 1291.
TE7.H5 no. 1291
[TE228.5]
388 s—dc20
[625.7] 91-8965
CIP

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Foreword

Too often low-volume roads are the last part of the road system to benefit from the latest engineering ideas. They seldom have been the object of research. Like poor cousins, they are too numerous and their problems too exacting for the limited resources of most of the world's major highway research agencies. Public works departments are conservative, and local roads agencies seem to be the most conservative and most poorly financed of all, so innovations often must wait until they are fully proven on major highways before they are used on low-volume roads.

To focus attention on this problem, in 1975 the first International Low-Volume Roads Conference was convened in Boise, Idaho, by the Transportation Research Board. The late Eldon Yoder of Purdue University provided the leadership to get the series of conferences under way. Since then, conferences have been held every 4 years. Their sites have included Ames, Iowa; Phoenix, Arizona; Ithaca, New York; and now Raleigh, North Carolina.

The conferences provide a forum that will move low-volume roads out of the poor cousin category. They have brought together major researchers and innovative practitioners to talk and to listen to each other. Whereas the conferences provide a meeting place for a few hundred people for a few days, these proceedings provide detailed communication to thousands of interested engineers and planners for many years following each conference.

These proceedings represent the combined effort of hundreds of people. Surely the greatest recognition should go to the authors of the papers printed in these two volumes. The credit for doing the research and documenting it is theirs alone, and we are deeply indebted to them. In addition to reviewing every detail of the conference plans, the conference steering committee spent several days reviewing synopses and choosing the papers that appear in these proceedings.

The papers underwent peer review. Nearly 500 reviewers offered suggestions on how the authors could improve their communications. A staff of editors at TRB spent many hours clarifying the language and putting the papers into standard form. Keeping track of all of the papers as they moved from initial synopsis to final manuscript was no small task, and credit for this goes to G. P. Jayaprakash and his secretaries at TRB.

None of this would have been possible without the financial support of the conference sponsors: the U.S. Department of Agriculture Forest Service, the Federal Highway Administration, the Bureau of Indian Affairs, and the U.S. Army Corps of Engineers. Though the conference participants paid the direct costs of the meeting in North Carolina, the sponsors funded the costs incurred by TRB for editing and printing the proceedings and publicizing, general management, and organization of the conference. Also, the conference hosts, the Institute for Transportation Research and Education of the University of North Carolina, organized the field trip and provided excellent facilities and staff support for the meeting.

To all of these many people I am deeply indebted. I extend my heartfelt thanks to each one.

Lynne H. Irwin
Steering Committee Chairman

Design and Construction

Determination of Line and Grade for New Low-Volume Roads: Implications of a Total-Cost Approach

NEVILLE A. PARKER

The selection of general alignment and gradient for a road is posed as a unique design decision driven by the objective of minimizing total life-cycle costs of construction and vehicle operation. Inputs to the design decision process include the terrain between the origin and destination; the roadway geometry and surface type; the vehicle volume, mix, and growth rate; unit construction and vehicle operating costs; design life; and interest rate. Outputs include combinations of horizontal alignments and piecewise gradients, representing various optima based on combinations of the life-cycle cost components. These outputs provide the basis for the subsequent design decisions. The analytical procedure includes a basic cost model that reduces the terrain to a number of grade-constrained construction surfaces by using linear programming and a route selection model that computes the life-cycle costs of various alternative alignments over the surfaces and selects the ones with the least cost by using shortest path and next-best path techniques. The implications of a total-cost approach for horizontal and vertical design standards are discussed. The overall implication, however, is the potential obsolescence of predetermined geometric design standards for other than urban roads and intersections, because these standards can be uniquely determined as outputs of an analytical process.

Socioeconomic and sociocultural development in developing countries mandates the expansion of networks by the addition of low-volume links as well as the upgrading of the network by realignment and relocation of major segments of existing links. Developed countries with their underdeveloped and developing hinterlands often face the same problems, and most certainly where forestry plays a significant role in the economy, new low-volume–heavy-axle-load roads must be built when logging shifts from one area to another.

The selection of line and grade before the detailed design of the roadway geometry and pavement structure has a profound impact on the total life-cycle cost of construction, vehicle operation, and maintenance. Therefore, the total life-cycle costs should be direct inputs to an analytical selection of line and grade as output, rather than the choice of line and grade by predetermined sets of standards, which is the conventional approach.

OBJECTIVES

The overall objective of this paper is to present an analytical approach to the selection of line and grade combinations unique to the particular location situation and most likely to result in minimum total life-cycle costs. In particular the following are discussed:

1. A basic cost model, which reduces the terrain in the zone of interest between termini of a location to a number of grade-constrained construction surfaces;
2. A route selection model, which uses the basic cost model to select optimum locations as a function of construction costs and the vehicle operation costs of fuel and oil consumption;
3. The implications of a total-cost approach for horizontal and vertical design standards for new and relocated low-volume roads; and
4. Extensions of these implications to selection of line and grade for nonurban highways in general.

PROBLEM-SOLVING APPROACH

The approach adopted in the paper is methodological. A 5-km direct distance (re)location situation is presented as an example, for which the terrain is known and digitized; the average daily traffic (ADT) at the opening of the road and the classified traffic growth rates are projected; and unit costs of construction and fuel-and-oil consumption are estimated for one surface type. Quantity relationships from the Road Transport Investment Model (RTIM2), as well as relationships developed by the author for construction, are utilized in the analytical procedure. Linear programming and shortest-path techniques are employed in the basic cost model and route selection model, respectively. The combined sensitivity of line and grade to the total-cost parameters of interest is demonstrated in the process.

The model described in this paper does not yet include maintenance as an explicit input to the determination of line and grade. The computer programs are also not user interactive. These and other graphical enhancements will be added to the conceptual model.

BASIC COST MODEL

The Basic Cost Model (BCM) is a trilevel model defined on a digitized search grid between an origin and a destination representing (a) the intervening terrain at the regularized grid points, (b) cuts and fills at the grid points, and (c) a smoothed

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construction surface on which an infinity of horizontal alignments may be defined.

In this model the construction surface is a polynomial. For any given maximum gradient, the polynomial surface is computed such that the sum of the absolute value of the differences between it and the terrain elevations at the grid points is a minimum, subject to the constraint that the first derivative of the surface function does not exceed the absolute value of the gradient (in either of the orthogonal directions) at the grid points. The differences represent fills (+) and cuts (−).

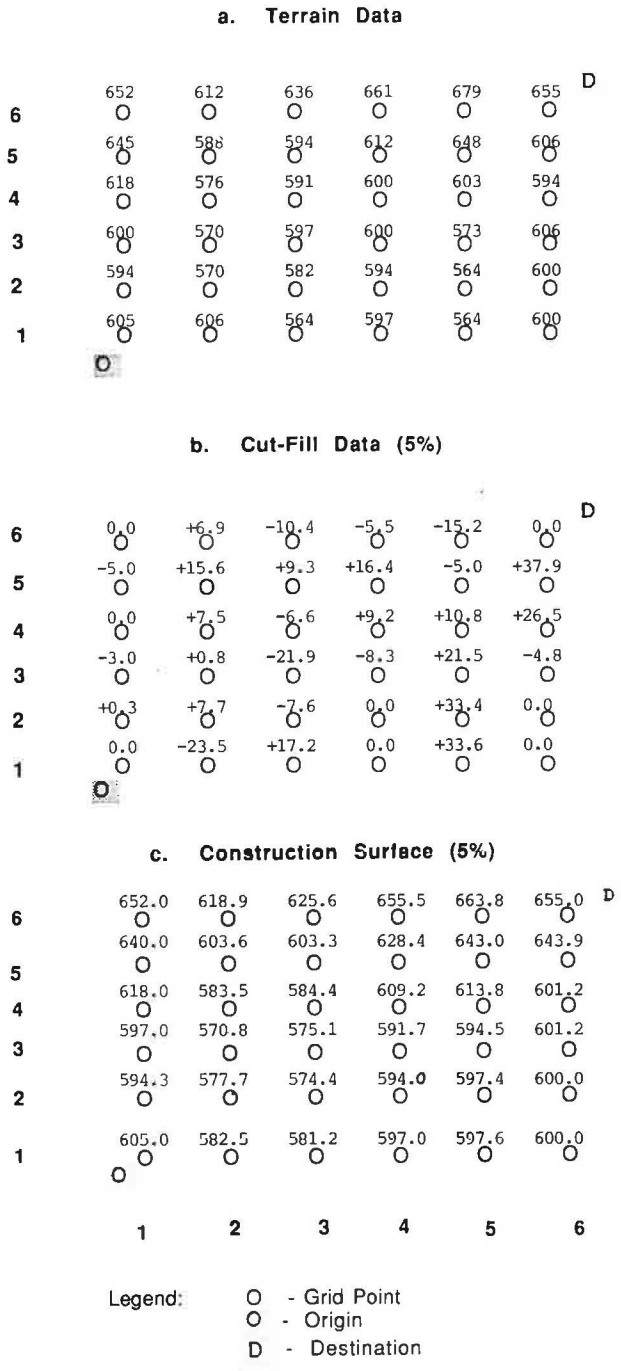


FIGURE 1 Basic Cost Model concept.

one or both of which will be zero, depending on whether the optimum polynomial surface lies above (+), below (−), or coincident with the terrain elevation at the grid point. The polynomial construction surface provides a rational basis for the estimation of the likely profiles of all possible O-D alignments through the grid points.

The BCM concept is shown in Figure 1. Sample terrain data digitized on a 6 × 6 grid between the origin (1,1) and the destination (6,6) are shown in Figure 1a. The optimum polynomial construction surface, digitized for a 5 percent maximum gradient, is shown in Figure 1c. Figure 1b is the difference between Figures 1c and 1a, showing the resultant cuts and fills. For example, the suggested optimum 5 percent alignment at grid point (4,4) with a terrain elevation of 600 m (Figure 1a) would necessitate a fill of 9.2 m.

The optimum polynomial construction surface function is a byproduct of the mathematical procedure adopted. Figure 2 shows how a linear programming formulation would yield a unique BCM for every assumed maximum gradient (g). The Z_{ij} are the terrain elevations of the grid points; end-point constraints may be omitted. The outputs of interest are the C_{ij} and F_{ij}, which are the cuts and fills, respectively, on the search grid.

The BCM forms the basis for the computation of all design-related costs.

ROUTE SELECTION MODEL

The Route Selection Model (RSM) consists of a minimum path algorithm, which defines a minimum path tree; a next-best path algorithm, which uses the concept of deviations to define as many best paths as needed; and a link-evaluation submodel, which defines the alternatives linkwise in terms of gradients, speeds, fuel consumption, construction costs, and so on.

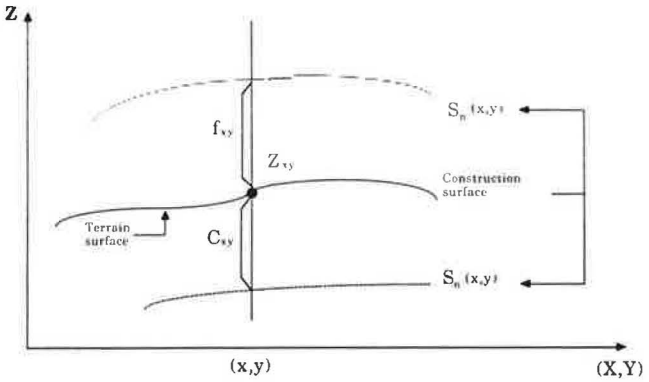
The link-evaluation submodel uses the BCM parameters along with other socioeconomic data. For example, construction quantities (to formation level) are estimated from the cuts and fills at the grid points and multiplied by unit costs to arrive at estimates of earthwork costs; from the construction surface piecewise gradients are computed and these are used, together with information about vehicle volume and mix, in the estimation of fuel and oil consumption (a major component of vehicle operation costs). Maintenance costs will also be influenced by gradients, particularly on unpaved roads in areas of heavy seasonal rainfall.

The cost estimates for vehicle operation are based on functional relationships developed by the U.K. Transport and Road Research Laboratory (TRRL) for RTIM2 (*1*). The procedure for the determination of line and grade is diagrammed in Figure 3. Conceptual and programming details may be found elsewhere (*2*).

SAMPLE PROBLEM

The grid in Figure 1 represents an area of search between an origin and destination 5 km apart. The objective is to search out, evaluate, and select economic locations in terms of both line and grade using life-cycle costs as a criterion.

Data related to construction and vehicle operation are shown in Figure 4 and Table 1. The type of surface is bituminous and the pavement width is 9.5 m. The design life is 15 years and the discount rate is 8.0 percent. The ADT on opening is given for each of five categories of vehicles, with their individual growth rates, fuel costs, and power-to-weight ratio (PW) and gross vehicle weight (GVW), where applicable.



Grid Point Function $Z_{ij} = S_n(i,j) + C_{ij} - f_{ij}$

MATHEMATICAL FORMULATION:

$$\text{MIN} \sum_i \sum_j C_{ij} + f_{ij}$$

Subject to:

general elevation constraints: $C_{ij} - f_{ij} + S_n(i,j) = Z_{ij} \quad \forall\, i,j$

general grade constraints:

$$\frac{\partial S_n(i,j)}{\partial x} \leq g \quad \forall_{i,j}$$

$$\frac{\partial S_n(i,j)}{\partial x} \leq g \quad \forall_{i,j}$$

end point elevation constraints:

$S_n(1,1) = Z_{1,1}$

$S_n(N,N) = Z_{n,n}$

end point grade constraints:

$$\frac{\partial S_n(1,1)}{\partial x} = \frac{\partial S_n(1,1)}{\partial x} = g_{1,1}$$

$$\frac{\partial S_n(N,N)}{\partial x} = \frac{\partial S_n(N,N)}{\partial y} = g_{N,N}$$

SURFACE FUNCTIONS : $S_n(x,y) = a_0(0) + a_1x + a_2y\,(1) + a_3x^2 + a_4xy^2 + a_5y^2(2) + a_6x^2 + a_7x^2y + a_8xy^2 + a_9y^3(3) + \cdots\cdots\cdots$

FIGURE 2 Linear programming formulation of the basic cost model.

Unit costs of construction (per cubic meter) to formation level are represented by cut-to-fill (CF-COST), cut-to-waste (CW-COST), and borrow-to-fill (BF-COST) at grid coordinates. Pavement unit costs (PV-COST) (per square meter) are likewise represented.

Uniform construction unit costs are used in order to isolate the effects of the interaction of traffic, terrain, and road geometry. Neither the effects of nor the impact on maintenance is included in the example. Thus locations may be selected on the basis of construction costs only, vehicle operating costs (fuel and oil) only, or construction plus vehicle operating costs. Referring to Figure 3, the procedure is as follows:

Step 1: Compute the BCMs for a range of maximum gradients (1.5, 2, 3, 4, 5, 6, 7, and 8 percent).

Step 2: Using each of the BCMs in turn with Figure 4 and Table 1, select sets of best paths on the cost basis mentioned above. Facsimiles of the output format are shown in Figures 5–7.

What is presented to the analyst is a wide range of alternative locations with their cost implications. In this example, for each gradient, six best paths are generated for each of the three cost bases, giving a total of 144 alternative optima. This number of optima constitutes a rational and extensive basis for a comparable evaluation and recommendation.

Comparisons of the first-minimum routes chosen on the basis of construction cost, vehicle operating costs, and the sum of construction plus vehicle operating costs are shown in Figures 8, 9, and 10, respectively. Figure 11 compares second-minimum routes using vehicle operating costs. Third, fourth, sixth, or any other level of minimum, could be arranged likewise for any cost combination. Note that by doing this, it is possible to identify some global optima and in the process determine a design maximum gradient.

IMPLICATIONS OF A TOTAL-COST APPROACH

Total-Cost Criterion

The total-cost approach can set the engineer free from the restriction of design standards and instead permit the exploration of a wide range and combination of design criteria (including surface type) in the search for an optimum that is acceptable to the particular society and its decision makers in terms of construction and vehicle operation (and maintenance) costs, as well as total costs.

In the latter respect, it is suggested here that total costs should be the first-level criterion to reduce the set of alternative optima. When applied to the sample problem, total costs would reduce the set of 144 (Table 2) to a subset of 18 (Table 3), including the least total cost for the first, second, and so on, up to the sixth-minimum routes, as determined by the three cost-based search criteria of construction, vehicle operation, and construction plus vehicle operation costs.

In the sample problem eight different gradients, from 1.5 (the lowest feasible gradient) to 8 percent, are analyzed. It is clear from Table 2, however, that the optimum gradient would most likely fall within the range of 4 to 6 percent. This conclusion is arrived at by noting that the minimum of the

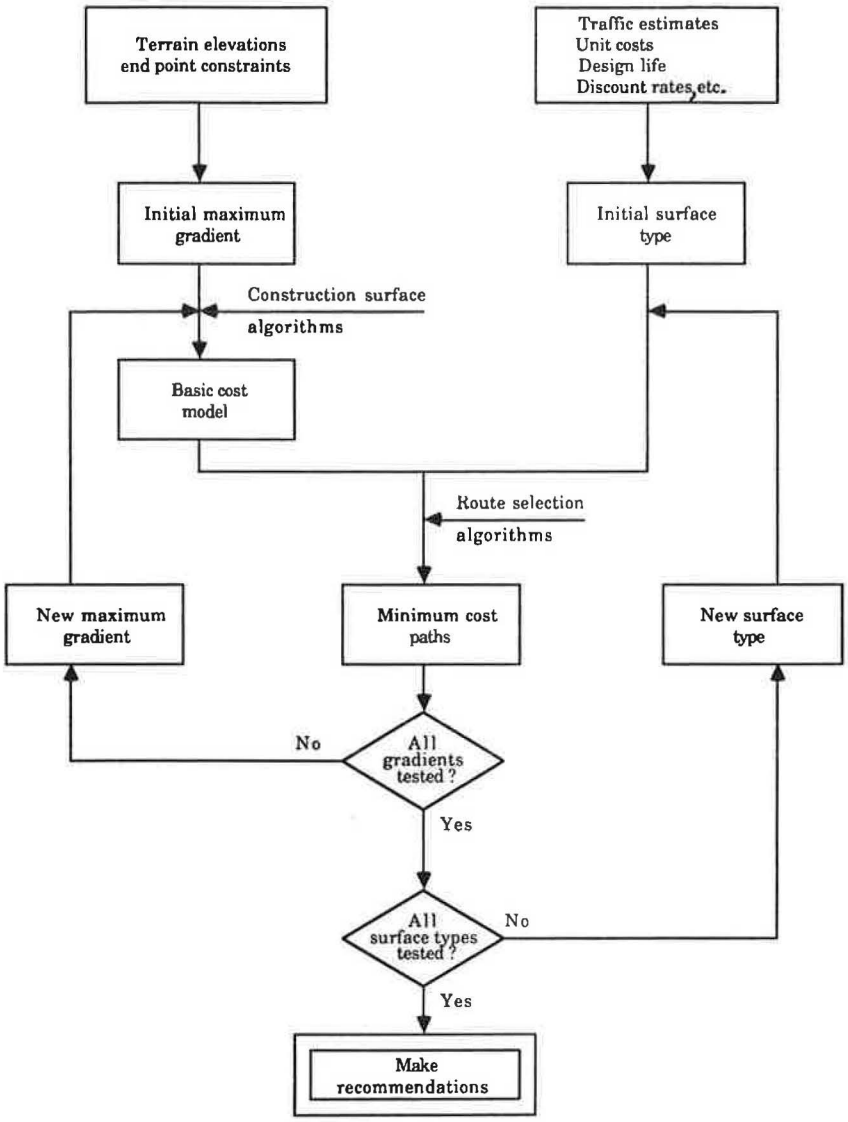


FIGURE 3 Procedure for determination of line and grade.

```
STRAIGHT-DISTANCE    = DISTANCE  5.0 KM
WIDTH OF PAVEMENT    = 9.5 M
TYPE OF SURFACE :    1.BITUMINOUS
MAX. GRADIENT = 5.0%
DESIGN LIFE    = 15 YEARS

DESIRED NO. OF BEST PATHS = 6

OPTIMISE ON TRAFFIC OPERATING COSTS ONLY

TRAFFIC DATA              ADT       GR(%)      F-COST     PW         GVW

1.PASSENGER CARS          150.      3.0        9.50
2.LIGHT GOODS VEHICLES     30.      3.0        9.50
3.MEDIUM GOODS VEHICLES    81.      3.0        6.50       30.0
4.HEAVY GOODS VEHICLES    350.      5.0        6.50       13.4       10.0
5.BUSES                   170.      4.0        6.50       10.0        8.5
  TOTAL                   781.      4.1
  DISCOUNT RATE                     8.0
  LUBR. OIL : UNIT COST                        4.50
```

FIGURE 4 Sample problem data.

TABLE 1 ZONE DATA AND CONSTRUCTION UNIT COSTS FOR SAMPLE PROBLEM

X	Y	ELEV.	CUT/FIL	SURFACE	CF-COST	CW-COST	BF-COST	PV-COST
1	1	605.0	0.0	605.0	22.50	22.50	27.50	500.00
2	1	606.0	-23.5	582.5	22.50	22.50	27.50	500.00
3	1	564.0	17.2	581.2	22.50	22.50	27.50	500.00
4	1	597.0	0.0	597.0	22.50	22.50	27.50	500.00
5	1	564.0	33.6	597.6	22.50	22.50	27.50	500.00
6	1	600.0	0.0	600.0	22.50	22.50	27.50	500.00
1	2	594.0	.3	594.3	22.50	22.50	27.50	500.00
2	2	570.0	7.7	577.7	22.50	22.50	27.50	500.00
3	2	582.0	-7.6	574.4	22.50	22.50	27.50	500.00
4	2	594.0	0.0	594.0	22.50	22.50	27.50	500.00
5	2	564.0	33.4	597.4	22.50	22.50	27.50	500.00
6	2	600.0	0.0	600.0	22.50	22.50	27.50	500.00
1	3	600.0	-3.0	597.0	22.50	22.50	27.50	500.00
2	3	570.0	.8	570.8	22.50	22.50	27.50	500.00
3	3	597.0	-21.9	575.1	22.50	22.50	27.50	500.00
4	3	600.0	-8.3	591.7	22.50	22.50	27.50	500.00
5	3	573.0	21.5	594.5	22.50	22.50	27.50	500.00
6	3	606.0	-4.8	601.2	22.50	22.50	27.50	500.00
1	4	618.0	0.0	618.0	22.50	22.50	27.50	500.00
2	4	676.0	7.5	583.5	22.50	22.50	27.50	500.00
3	4	591.0	-6.6	584.9	22.50	22.50	27.50	500.00
4	4	600.0	9.2	609.2	22.50	22.50	27.50	500.00
5	4	603.0	10.8	613.8	22.50	22.50	27.50	500.00
6	4	594.0	26.5	620.5	22.50	22.50	27.50	500.00
1	5	645.0	-5.0	640.0	22.50	22.50	27.50	500.00
2	5	588.0	15.6	603.6	22.50	22.50	27.50	500.00
3	5	594.0	9.3	603.3	22.50	22.50	27.50	500.00
4	5	612.0	16.4	628.4	22.50	22.50	27.50	500.00
5	5	648.0	-5.0	643.0	22.50	22.50	27.50	500.00
6	5	606.0	37.9	643.9	22.50	22.50	27.50	500.00
1	6	652.0	0.0	652.0	22.50	22.50	27.50	500.00
2	6	612.0	6.9	618.9	22.50	22.50	27.50	500.00
3	6	636.0	-10.4	625.6	22.50	22.50	27.50	500.00
4	6	661.0	-5.5	655.5	22.50	22.50	27.50	500.00
5	6	679.0	-15.2	663.8	22.50	22.50	27.50	500.00
6	6	655.0	0.0	655.0	22.50	22.50	27.50	500.00

optima in Table 2 (i.e., row-wise) fall between 4 and 6 percent. One might go even further to conclude that the 5 percent gradient is the design gradient that would be most likely to produce the minimum total cost for a wide range of options, given that it produced the minimum in two-thirds of the optima within the 4 to 6 percent range and that it completely dominates the range of gradients for the total-cost (construction plus vehicle operating costs) criteria. The 5 percent gradient is also seen to produce the global minimum among the optima.

Construction Cost Versus Vehicle Operation Cost Criterion

The total-cost criterion was used to reduce the set of 144 optima in Table 2 to the 18 optima in Table 3. Table 3 also displays additional details concerning approximate route length design, maximum and average gradients, and the breakdown of total costs into those for construction and those for vehicle operation. This subset of minima is also rank ordered from 1 to 14 (only 14 because of duplication of routes).

Construction costs would clearly dominate the engineering decision-making process in this case by the model used. The minimum-cost construction route also produces the minimum total-cost route. By contrast, the twelfth-ranked alternative produces the lowest vehicle operating cost but at the expense of almost 2.5 times the construction cost of the least-construction-cost alternative. Clearly this would be difficult for the ultimate decision maker (who would also be the ultimate financier) to accept. The long-term benefit of lower fuel consumption might not appear to justify such a substantial differential in immediate expenditure. The sixth-ranked alternative just might be acceptable in discussion, but is not likely to be adopted either. In general, however, following the narrowing down of the infinite number of alternatives to a finite subset of total-cost optima, this subset is further subjected to an analysis in terms of the trade-offs between constituent costs (in this case, construction and vehicle operation costs).

Alignment and Gradient

The design problem may be restated as the simultaneous selection of the general alignment and gradient most likely to result in the minimum life-cycle costs subject to acceptable or financially feasible levels of (initial) construction costs and

OUTPUT FOR N-BEST PATHS

PATH NUMBER 1 FROM ORIGIN (1,1) TO DESTINATION : (6,6)

X	Y	FLEV	CUT/FILL	SURFACE	GRADE	COST("000)
6	6	655.	0.00	655.00	1.2	25403.758
5	5	648.	-5.00	643.00	3.4	20528.919
4	4	600.	9.20	609.20	3.4	15218.693
3	3	597.	- 21.90	575.10	-0.3	9897.609
2	2	570.	7.70	577.70	-2.7	5180.740
1	1	605.	0.00	605.00	0.0	0.000

APPROX. ROUTE LENGTH = 4.999 KM MAX. GRADE = 3.4 % AVG. GRADE = 2.2%

CONSTRUCTION COST = 30763.332
OPERATING COST = 25403.758
TOTAL = 56167.090

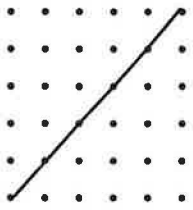


FIGURE 5 Facsimile output for vehicle operating costs: Path No. 1.

PATH NUMBER 2 FROM ORIGIN (1,1) TO DESTINATION : (6,6)

X	Y	FLEV	CUT/FILL	SURFACE	GRADE	COST('000)
6	6	655.	0.00	655.00	1.2	26904.584
5	5	648.	-5.00	643.00	3.4	22029.744
4	4	600.	9.20	609.20	2.5	16719.519
4	3	600.	-8.30	591.70	1.7	13298.322
3	2	582.	-7.60	574.40	-0.5	8424.784
2	2	570.	7.70	577.70	-2.7	5180.740
1	1	605.	0.00	605.00	0.0	0.000

APPROX. ROUTE LENGTH = 5.414 KM MAX. GRADE = 3.4 % AVG. GRADE = 2.1%

CONSTRUCTION COST = 12474.325
OPERATING COST = 26904.584
TOTAL = 39328.909

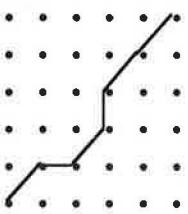


FIGURE 6 Facsimile output for vehicle operating costs: Path No. 2.

PATH NUMBER 3 FROM ORIGIN (1,1) TO DESTINATION : (6,6)

X	Y	FLEV.	CUT/FILL	SURFACE	GRADE	COST('000)
6	6	655.	0.00	655.00	1.2	26911.263
5	5	648.	-5.00	643.00	4.1	22036.423
5	4	603.	10.80	613.80	2.2	18373.540
4	3	600.	-8.30	591.70	1.7	13298.322
3	2	582.	-7.60	574.40	-0.5	8424.784
2	2	570.	7.70	577.70	-2.7	5180.740
1	1	605.	0.00	605.00	0.0	0.000

APPROX. ROUTE LENGTH = 5.414 KM MAX. GRADE = 4.1 % AVG. GRADE = 2.1%

CONSTRUCTION COST = 13288.566
OPERATING COST = 26911.263
TOTAL = 40199.829

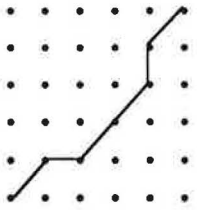


FIGURE 7 Facsimile output for vehicle operating costs: Path No. 3.

Surface Gradient	Best Path Length	Max. Grade	Average Grade	Construction Cost	Operating Cost	Total Cost
1.5	7.242	1.1	0.7	58 256 898	63 369 816	121 626 703
2.0	1.242	1.5	0.7	45 157 879	63 459 570	108 617 437
3.0	5.414	2.9	1.5	13 064 119	49 036 852	62 100 969
4.0	1.242	3.0	1.2	15 961 186	64 750 578	80 741 168
5.0	5.414	3.5	2.2	7 975 046	50 530 883	58 505 930
6.0	5.828	3.9	1.8	6 096 197	53 767 906	61 864 102
7.0	5.828	6.7	2.4	7 662 651	55 203 457	62 866 109
8.0	5.828	6.7	2.3	7 379 526	55 139 320	62 518 848

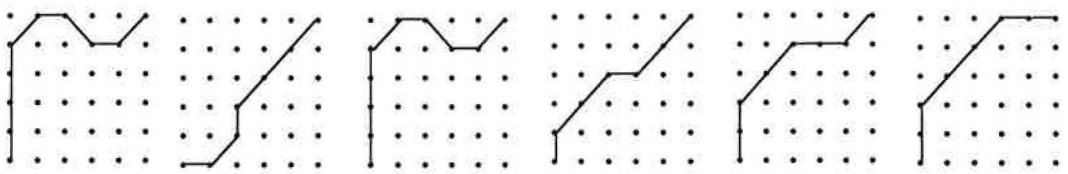
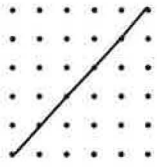


FIGURE 8 First-minimum construction cost routes for sample problem.

Surface Gradient	Best Path Length	Max. Grade	Average Grade	Construction Cost	Operating Cost	Total Cost
1.5	5.000	1.6	1.0	159 861 078	45 015 461	204 876 539
2.0	5.000	2.1	1.1	94 114 133	45 165 156	139 279 289
3.0	5.000	2.9	1.5	37 451 848	45 957 336	83 409 180
4.0	5.000	3.1	1.8	19 298 033	46 493 078	65 791 111
5.0	5.000	3.4	2.2	30 763 340	47 238 242	78 001 582
6.0	5.000	3.5	2.2	34 601 477	47 240 188	81 841 665
7.0	5.000	3.5	2.1	33 006 930	47 080 773	80 087 703
8.0	5.000	3.5	2.1	26 662 299	46 956 137	73 618 436



(all gradients)

FIGURE 9 First-minimum vehicle operating cost routes for sample problem.

Surface Gradient	Best Path Length	Max. Grade	Average Grade	Construction Cost	Operating Cost	Total Cost
1.5	7.242	1.1	0.7	58 256 898	63 369 816	121 626 704
2.0	5.414	2.1	1.2	48 180 902	48 532 527	96 713 429
3.0	5.414	2.9	1.5	13 064 119	49 036 852	62 100 971
4.0	5.000	3.1	1.8	19 298 033	46 493 078	65 791 111
5.0	5.414	3.5	2.2	7 975 046	50 530 883	58505 929
6.0	5.828	3.9	1.9	8 096 197	53 767 906	61 864 103
7.0	5.828	6.7	2.4	7 662 651	55 203 457	62 866 108
8.0	5.828	6.7	2.3	7 379 526	55 139 320	62 518 846

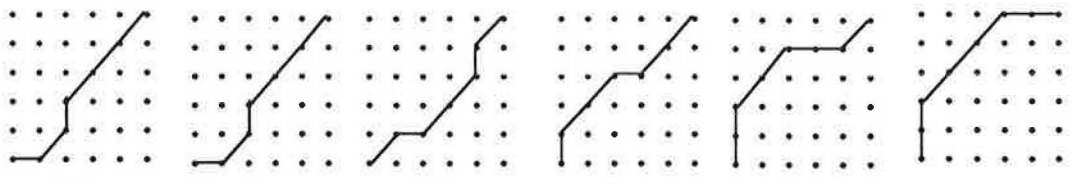


FIGURE 10 First-minimum total-cost routes for sample problem.

Surface Gradient	Best Path Length	Max. Grade	Average Grade	Construction Cost	Operating Cost	Total Cost
1.5	5.414	1.5	0.9	205 191 016	47 754 113	252 945 111
2.0	5.414	2.0	1.0	118 218 797	47 818 082	166 136 859
3.0	5.414	2.5	1.3	78 729 164	48 540 324	127 269 492
4.0	5.414	3.2	1.7	33 191 858	49 341 195	82 533 055
5.0	5.414	3.4	2.1	12 474 326	50 032 426	62 506 758
6.0	5.414	4.4	2.0	35 524 254	50 078 895	85 603 148
7.0	5.414	4.3	2.0	32 251 336	49 919 668	82 171 000
8.0	5.414	3.5	2.0	33 489 569	49 821 398	83 310 969

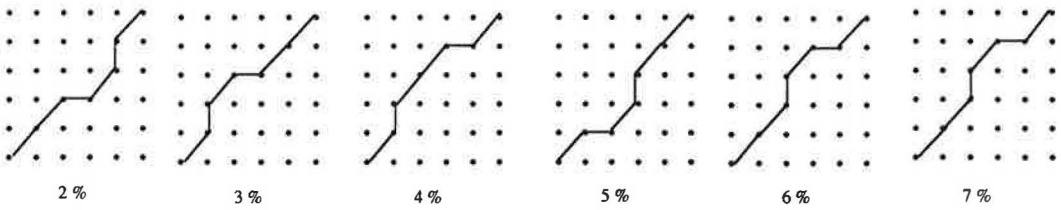


FIGURE 11 Second-minimum vehicle operating cost routes for sample problem.

(future) vehicle operation (and maintenance) costs. But what are the real design implications for alignment and gradient?

From Table 3, 5 percent would appear to be the most economical overall design gradient to adopt. It may be said, therefore, that one of the design outputs from the process is the overall design gradient, 5 percent. One would note, however, that the maximum gradient among the alternative optima need not necessarily equal the overall design gradient and that the average gradient over the alignment (i.e., the algebraic sum of all the rises and falls along the alignment divided by the length) is less than the maximum gradient, as is to be expected.

In the sample problem, route lengths vary from the direct distance of 5 km to approximately 6.4 km. There are three factors to note here, namely,

1. Construction cost considerations favor curvilinear alignments the extent of which is a function of overall design gradient;
2. Vehicle operating cost considerations favor direct distance alignments, regardless of gradient; and
3. Simultaneous consideration of both construction and vehicle operating costs moderates the extremes of curvilinearity and direct distance alignments.

It should also be noted with respect to the first factor that even when construction cost is the only consideration, low unit cost of construction should be traded off against length of alignment so that the total cost of construction does not exceed some maximum or concept of an optimum. One might consider, therefore, that construction cost considerations establish the maximum length, and hence curvilinearity. On the other hand, the impetus to minimize operating cost via a direct distance route should dictate that the alignment selection process be driven by a total-cost comparison of incrementally more curvilinear alignments with the direct distance route. Thus, all other locations would be viewed as "deviations" from the direct distance route to reduce construction costs to an acceptable maximum.

Finally, it should be noted that the different alignments, even for the same route lengths, influence costs through the piecewise combinations of gradient and curvature. That is to say, the design output from the process should also specify the maximum, average, and piecewise gradients (see Figures 5–7) as well as the overall design gradient for the recommended alignment if the order of magnitude of the cost implications is to be obtained. The importance of specifying all these aspects of gradients can be more readily appreciated by recalling that for any given alignment there are an infinite number of combinations of piecewise gradients, and hence an infinite set of cost implications. Thus, if a particular cost combination is deemed acceptable at a preliminary design stage (e.g., location design), it follows that the combination of alignment and gradients that produced the acceptable results must also be specified as a guide and comparator for any subsequent refinements or adjustment to the alignment and gradient.

Surface Type

The sample problem assumed a bituminous type of road surface. The analysis could be repeated for alternative surfaces to test the sensitivity of the general alignment and gradient, and to expand the range of optima, and hence the decision-making base. Essentially the model would trade off the low costs of construction for lower surface types against the higher costs of vehicle operation. Lower surface types might, how-

TABLE 2 OPTIMUM TOTAL-COST ROUTES FOR SAMPLE PROBLEM

Search criteria	Route designation	Total costs of construction and vehicle operation							
		1.5%	2.0%	3.0%	4.0%	5.0%	6.0%	7.0%	8.0%
Construction costs	1st minimum	121626703	108617437	62100969	80741766	**58505930**	61864102	62866109	62518848
	2nd minimum	130968117	118163047	62657523	78936055	**61573219**	66575727	67836789	67585047
	3rd minimum	130777414	96713438	66196961	74282125	73396805	**62100629**	66516078	65941844
	4th minimum	140019828	109187844	66094766	79686773	**62299230**	66965836	76750773	78677445
	5th minimum	140076500	102743289	74700672	82582117	67342148	**65554430**	78595312	76917102
	6th minimum	149318906	109706258	71818094	82985531	71477664	**66812258**	69162438	78033438
Vehicle operation costs	1st minimum	204876562	139279281	83409180	**65791109**	78001578	81841664	80087703	73618437
	2nd minimum	252945141	166136859	127269492	82533055	**62506758**	85603148	82171000	83310969
	3rd minimum	241681953	145293812	127106781	78724281	**63333465**	86802477	85111445	75099055
	4th minimum	231689531	215761172	111500320	80596883	86184281	**65019020**	69881344	85191828
	5th minimum	226929906	211381062	91538328	**73383766**	79755898	85651805	86773477	83505547
	6th minimum	285017437	265746469	88947773	68802453	80681812	**64798516**	80946242	77950133
Construction + vehicle operation costs	1st minimum	121626703	96713438	62100969	65791109	**58505930**	61864102	62866109	62518848
	2nd minimum	130777414	98751539	62657523	68075750	**61573219**	62100629	63541559	63074852
	3rd minimum	130869117	102743289	66094766	68345133	**61818090**	62908980	63627457	64097785
	4th minimum	138764094	102953648	66196961	68802453	**62299230**	64090934	64302906	64653789
	5th minimum	140019828	108617437	66819820	69686820	**62506758**	64798516	64932250	64686043
	6th minimum	140019828	109187844	67338086	69964609	**62843051**	65019020	65693602	65242047

TABLE 3 TOTAL-COST ARRAY FOR SAMPLE PROBLEM

Search criteria	Route designation	Route length (km)	Gradients (%)			Costs			Rank order
			design	max.	average	construction	operation	total	
Construction costs	1st minimum	5.414	5.0	3.5	2.2	**7975046**	50530883	**58505929**	1
	2nd minimum	5.828	5.0	3.5	1.7	8200056	53373160	61573216	2
	3rd minimum	5.828	6.0	3.9	2.1	8357019	53743609	62100628	4
	4th minimum	5.828	5.0	3.5	2.0	8610596	53688633	62299229	5
	5th minimum	6.242	6.0	4.1	2.0	8384327	57170102	62554429	11
	6th minimum	6.413	6.0	3.5	1.9	8391141	58421117	66812258	13
Vehicle operation costs	1st minimum	5.000	4.0	3.1	1.8	19298033	**46493078**	65791111	12
	2nd minimum	5.414	5.0	3.4	2.1	12474325	50032426	62506751	6
	3rd minimum	5.414	5.0	4.1	2.1	13288567	50044895	65333462	8
	4th minimum	5.414	6.0	3.5	2.1	14860564	50158453	65019017	10
	5th minimum	5.414	4.0	3.1	1.8	23945572	49438191	73383763	14
	6th minimum	5.414	6.0	4.5	2.1	14597288	50201223	64798511	9
Construction + vehicle operation costs	1st minimum	5.414	5.0	3.5	2.2	**7975046**	50530883	**58505929**	1
	2nd minimum	5.828	5.0	3.5	1.7	8200056	53373160	61573216	2
	3rd minimum	5.414	5.0	3.5	2.2	11497665	50320422	61818087	3
	4th minimum	5.828	5.0	3.5	2.0	8610596	53688633	62299229	5
	5th minimum	5.414	5.0	3.4	2.1	12474325	50032426	62506751	6
	6th minimum	5.828	5.0	3.7	2.1	8844034	53999016	62843050	7

ever, lead to higher costs for improved geometric alignment in order to lower operating costs.

Design Standards

Location design models can be used to establish and revise standards appropriate to any particular physical and socioeconomic environment, despite the above assertion that predetermined design standards are not a necessary requirement for the initiation of the design process. For example, the procedure may be used for a range of vehicle volumes and mix over different terrain types in a given country with its particular factor inputs, the objective being to establish threshold values as guides for the conventional highway design process. In the sample problem the likely range of gradients was narrowed down to between 4 and 6 percent. This approach could be expanded and generalized to facilitate the development and evolution of design guides.

CONCLUSIONS

The results presented in this paper suggest that

1. The economics of low-volume road design standards can be improved by sophisticated analytical methods,
2. Linkwise line and grade combinations can be uniquely determined as design output rather than standard input for a given (re)location problem, and
3. There are implications for setting highway design standards that are appropriate to specific socioeconomic environments.

Low-volume roads do not necessarily trigger low design standards, such as steep gradients, high curvature, and low riding surfaces. When the vehicle operating costs are included, these moderate the conventional response toward longer alignments with low unit construction costs to shorter alignments with higher unit construction costs but lower total construction costs. These trade-offs would be difficult to evaluate on a project-by-project basis without an analytical process that can simultaneously select line and grade on a total-cost minimization basis.

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Factors Affecting Maximum Gradeability of a Log Truck Around a Curve

PAUL ANDERSON AND JOHN SESSIONS

Steep road grades provide managers with a way to reduce the economic and environmental costs of transportation systems. Available formulas for calculating log truck gradeability do not consider the performance of log trucks on horizontal curves. The maximum grade that a log truck can climb on a circular curve is less than that on a tangent for a number of reasons. These factors are discussed and some preliminary results obtained from a mathematical model are presented. Predictions from the mathematical model are compared with field observations collected from a survey of steep road operations from forest road managers throughout the USDA Forest Service.

During the last 20 years there has been increasing concern within the forest community about road-related landslides, impacts of roads on visual quality, and increased road construction costs. Permanent gravel-surfaced roads with grades up to 20 percent and temporary unsurfaced roads up to 26 percent have been negotiated successfully by loaded and unloaded log trucks under their own power. One critical element in evaluating feasibility of such steep, adverse grades is estimating the maximum grade that a loaded log truck-trailer combination can climb at a constant speed without losing traction. This maximum grade is referred to as the traction-limited gradeability for the vehicle, or often simply as the gradeability. Modern logging trucks often have engines of 350 to 400 hp with sufficient torque in low gear so that their grade-climbing ability is not limited by torque requirements. Formulas for evaluating the maximum traction-limited gradeability for loaded and unloaded log trucks on tangent road sections with no superelevation have been derived (*1*). These formulas consider rolling resistance, vehicle geometry, weight distribution between axles, and the coefficient of traction between the tires and the running surface.

The maximum grade that a log truck can climb on a circular curve is lower than that on a tangent due to a number of factors. Many road managers recognize that gradeability around a curve is lower and use various rules of thumb to reduce the maximum design grade around the curve relative to the tangent-limited grade. Factors that reduce gradeability around horizontal curves are discussed and predictions of a mathematical model are compared with results from a survey of steep grade operations taken from road managers throughout the USDA Forest Service.

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FACTORS AFFECTING GRADEABILITY AROUND CURVES

Unlike log trucks on tangents, log trucks on horizontal curves are affected by six factors that do not exist on tangents:

1. Actual road grades (effective grades) for the tractor and trailer that differ from the centerline grade,
2. Tandem drag,
3. Resisting forces on the log load and trailer that do not act parallel to the truck-tractor,
4. Centrifugal force,
5. Superelevation, and
6. Torque requirements of the drive axle differential.

The vehicle referred to in this discussion is the typical truck-tractor and pole-steered trailer used throughout western North America (Figure 1). The trailer is steered by a telescoping pole that connects the trailer axles to the frame of the truck-tractor. The telescoping pole is not designed to be in tension and the trailer is pulled by the logs held by friction between the logs and the log bunks. The pole-steered trailer requires less road width than the conventional trailer steered by the fifth wheel (*2*).

Effective Grades

As a loaded logging truck travels around a curve on a grade, the grade along the centerline of the truck and trailer is different from the grade along the centerline of the road because the truck and trailer tandem axle sets do not follow the path of the steering wheels around the curve. In effect, the trailer straddles the roadway. This straddling of the roadway creates a steeper grade (the effective grade) than the trailer would have experienced on a tangent (Figure 2), making the trailer drag due to gravity higher than would have otherwise occurred.

Tandem Drag

As a tandem axle is pulled around a curve, drag forces are created because of the turning resistance of the tandem axles (*3*). This drag resistance is proportional to the normal load on the axles and inversely proportional to the curve radius. Tandem drag does not exist on a tangent and must be added both to the forces acting on the truck-tractor because of the drag of the tractor drive tandems and to the trailer if both units have tandem axles.

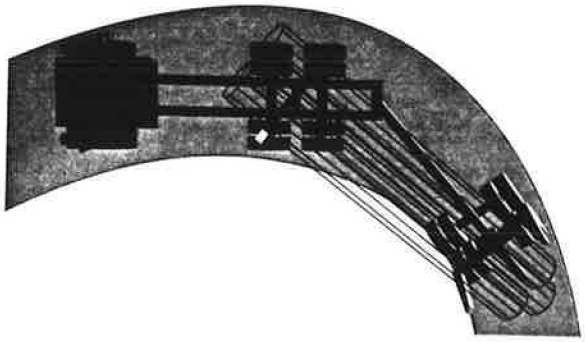

FIGURE 1 Perspective view looking down on a loaded log truck-tractor with pole-steered trailer climbing around a steep curve.

Nonparallel Forces

Because of the turning action of the truck and trailer on a curve, the pull of the log load transmitted by the logs to the front bunks is not on a line parallel to the truck-tractor (Figure 3). This creates a moment about the drive axles that redistributes the wheel loading from what would have occurred on a tangent. The normal load on the outside wheels is reduced and the loading on the inside wheel is increased.

Centrifugal Force

Centrifugal force arises from acceleration due to the changing direction around the curve. Centrifugal force acts through the center of mass and is proportional to the square of the velocity and inversely proportional to the curve radius. The effect of centrifugal force is to unload the inside wheels and load the outside wheels.

Superelevation

Superelevation creates a redistribution of the wheel loadings by introducing a gradient perpendicular to the centerline gradient. Superelevation can either load or unload the inside (or outside) wheels depending upon the direction of the superelevation. If the road is sloped inward toward the curve center (positive superelevation), the normal force on the inside wheels is increased and that on the outside wheels is reduced. Higher-speed roads normally have positive superelevation to counteract the overturning or sliding effects of centrifugal force. Superelevation also affects the effective grades of the tractor and trailer straddling the curve.

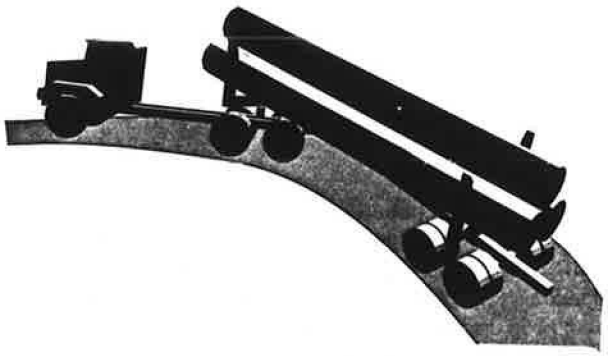

FIGURE 2 Inside-of-curve perspective view of a loaded log truck climbing around a steep curve.

FIGURE 3 Outside-of-curve perspective view of a loaded log truck climbing around a steep curve.

Axle Differential

Logging trucks are equipped with differentials between the drive axle shafts to permit the inside wheels to turn at a slower speed than the outside wheels as the truck goes around a curve. While the differential gears are operating (unlocked), the torque transmitted to each axle by the drive shaft is the same. Effectively, this means that the maximum thrust for the axle (left and right wheel sets combined) is equal to two times the most lightly loaded side.

If the traction coefficients under each wheel set are the same and the tire radii are equal, the greatest combined thrust can only occur when the normal forces on the left and right wheel sets are equal. If wheel loading is unequal and torque requirements are high, the lightly loaded wheel cannot absorb the same torque as the more heavily loaded wheel and will begin to spin. Because of differential action, as the lightly loaded wheel increases its revolutions per minute, the revolutions per minute of the opposite wheel are reduced and eventually stop. Limited slip differential devices can maintain torque to the more heavily loaded wheel set, but gradeability is still less than that on tangents because of the lower ratio of normal force on the driving wheels to gross vehicle weight.

MODELING GRADEABILITY AROUND CURVES

A model has been developed to calculate the maximum grade that a log truck could climb around a curve, considering the

six factors discussed previously. Using an iterative algebraic procedure, the procedure begins with guessing a maximum centerline grade and comparing the available thrust at the driving wheels with the thrust required to balance the resisting forces. The algorithm is terminated when the grade is identified at which the maximum available thrust at the driving wheels equals the required thrust to negotiate the grade. The nine steps in the algorithm are as follows:

Step 1: Establish the location of the truck and trailer wheels on the curve. For trucks on long curves, the equation from Anderson et al. (*4*) can be used. For trucks on short curves where full offtracking of the trailer wheels is not developed, the procedures outlined by Erkert et al. (*2*) can be used.

Step 2: Calculate the effective grades for the truck-tractor and the loaded trailer given the wheel locations calculated in Step 1.

Step 3: Solve for the normal force on the trailer axle and the reactions at the front log bunk pin (similar to fifth-wheel location) created by the loaded trailer. These are calculated by summing forces parallel and perpendicular to the effective grade of the trailer calculated in Step 2 and a moment balance around the midpoint of the trailer axles.

Step 4: Resolve the reactions at the front bunk calculated from Step 3 perpendicular and parallel to the effective grade of the truck-tractor that was calculated in Step 2.

Step 5: Calculate the centrifugal force on the driving wheels for an assumed truck velocity.

Step 6: Calculate the normal force on the combined left and right driving wheels considering the forces acting from the trailer (Step 4), the center of gravity of the truck-tractor, and rolling resistance of the truck-tractor.

Step 7: Calculate the normal force on the inside and outside wheel sets considering the trailer reactions on the front bunk (Step 4), centrifugal force on the driving wheels (Step 5), and the normal force on the combined left and right driving axles (Step 6).

Step 8: Compare the thrust available at the driving wheels with the vector sum of the thrust required to oppose the forces parallel and perpendicular to the driving wheels. The traction-limited thrust available at the driving wheels is equal to two times the normal force on the most lightly loaded side multiplied by the coefficient of traction.

Step 9: If the thrust available at the driving wheels exceeds the thrust required to overcome the resisting forces, increase the centerline grade and return to Step 1. If the available thrust is less than the sum of the resisting forces, reduce the centerline grade and return to Step 1. A numerical method such as binary search or the secant method can be used to identify the maximum centerline grade within a few iterations.

AN EXAMPLE

Using the gradeability algorithm, specifications for a typical loaded log truck with pole-steered trailer, a truck speed of 3.5 mph, and a coefficient of traction of 0.45, the gradeability was calculated for various curve radii and superelevation rates. A speed of 3.5 mph was chosen as being representative of the maximum speed in low gear. The net engine power requirement is on the order of 220 to 300 hp depending upon the final gradeability. The coefficient of traction of 0.45 was chosen as representative of the lower limit for firm native soil or medium-packed gravel (*1,5*). In order to simplify the presentation, the log truck was assumed to be in a deep curve, that is, a curve long enough for maximum offtracking to have been developed. This approach is a conservative one that provides the lowest estimate of gradeability.

Superelevation was varied from −6 to +6 percent (Figure 4). Log truck gradeability increases with curve radius. At positive superelevation, gradeability was reduced as compared with no superelevation because of the low centrifugal forces generated at the assumed truck speed of 3.5 mph. Higher truck speeds (up to a point) would have permitted higher gradeability, but it was believed that assuming higher speeds on steep grades was a less conservative approach.

With negative superelevation, log truck gradeability increased when compared with no superelevation. With a −6 percent superelevation, maximum gradeability increased until the curve radius became larger than 100 ft and then decreased. This decrease in gradeability is due to an excessive amount of negative superelevation. At larger curve radii, the trailer tracks

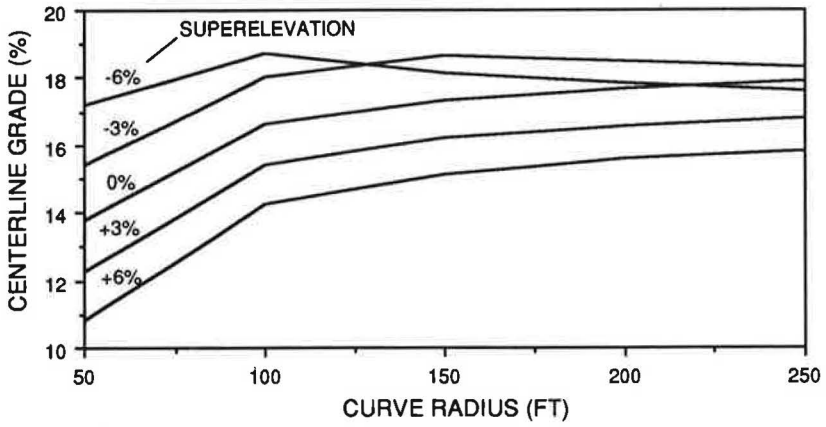

FIGURE 4 Gradeability as a function of curve radius and superelevation for a coefficient of traction of 0.45.

more closely with the truck-tractor, reducing the angle of pull of the trailer. The reduced angle of pull reduces the inward force that the combined effects of centrifugal force plus negative superelevation counteract.

COMPARISON OF THEORETICAL WITH REPORTED PERFORMANCE

To identify the experience of road managers with operation of roads in steep terrain, a survey of USDA Forest Service road managers was conducted during 1988 on all national forests in the western United States. Road managers were asked to document specific road projects with steep roads, citing location, surface type, grade in direction of loaded haul, curve radius, and superelevation. Responses totaled 107 ranging from successfully negotiated favorable grades of −35 percent to successfully negotiated adverse grades of +26 percent. Of the 37 reports of adverse grade operations on curves with no superelevation, 30 were negotiated successfully and 7 were not. Successes ranged from a 26 percent grade on a 285-ft-radius curve on a native material surface to a 12 percent grade on a 50-ft-radius curve, also on native material. The highest of the reported successful steep road operations on roads with no superelevation are plotted in Figure 5. Several reports were available for trucks operating on steep grades with horizontal curves and negative superelevation. The reports for a 50-ft radius and a 250-ft radius were for a −3 percent superelevation and the 100-ft radius was for a −4 percent superelevation (Figure 6). All three reports were for operating on native surface.

SUMMARY

Theoretical analysis and operational experience suggest that the gradeability of a log truck around a curve is less than that on a tangent section. A necessary condition for maximum gradeability of a log truck with trailer is to have equal normal force on the left and right wheel sets of the driving wheels. Road designers recognizing this principle can adjust road designs to achieve this force balance. Four design factors are within

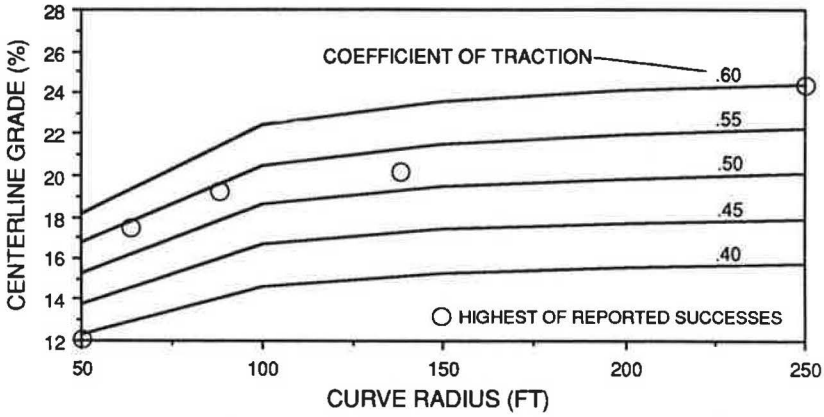

FIGURE 5 Gradeability as a function of traction coefficient, curve radius, and zero superelevation. Points of highest reported successful operations are identified.

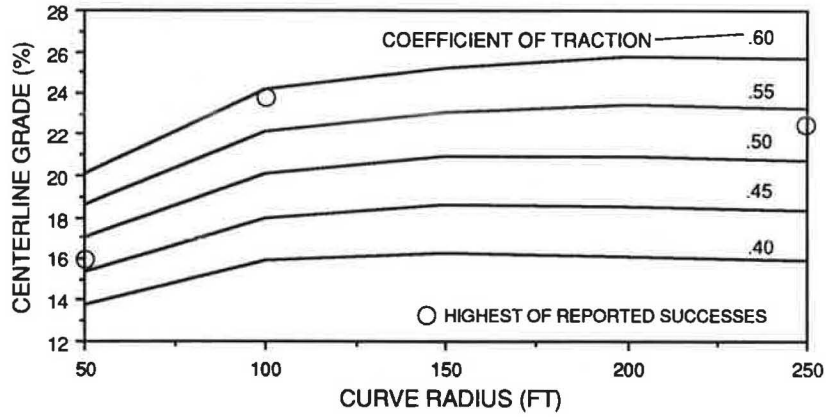

FIGURE 6 Gradeability as a function of traction coefficient, curve radius, and −3 percent superelevation. Points of highest reported successful operations are identified.

control of the road designer: centerline grade, road surface, curve radius, and superelevation. Understanding the interaction of these variables can provide more flexibility in road design and location.

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Full-Depth Reclamation of Asphalt Roads with Liquid Calcium Chloride

JAMES B. PICKETT

By reusing the asphalt from reconstruction of deteriorating roads and blending it with the gravel base and liquid calcium chloride, states, cities, and towns are able to rebuild their roads at a 50 percent savings. The implementation of a full-depth reclamation program with calcium chloride is a workable, cost-effective solution to the deterioration of roads. Many asphaltic roadways are in advanced stages of deterioration from aging, base problems, and drainage. Some roads have been maintained with the application of a periodic seal coat or an overlay. Because overlays last only a limited time before cracking begins to show through and because of insufficient funding to completely reconstruct the roadways, full-depth reclamation with calcium chloride is helping throughout the country.

Full-depth reclamation is a method that shows what recycling is all about—savings. There are savings in time, savings in materials, and savings in the use of additives.

Full-depth reclamation is a technique in which the full flexible pavement structure and a predetermined portion of the underlying base materials are uniformly crushed, pulverized, blended, or sized to result in a stabilized base course. This method significantly extends the life of the road and the amount of road work done for a budgeted dollar. It is with this objective in mind that calcium chloride is used as a stabilizer by recycling it with asphalt surfaces and base materials from reconstruction. A combination of crushed surface and base material is mixed with calcium chloride to facilitate the road builder's oldest form of stabilization: thorough mix and uniform compaction.

In the field of highway engineering stabilization is recognized as including all procedures for improving the performance of the asphalt, soils, and aggregates used in road construction and maintenance. A highway material may be considered stable if it exhibits a high degree of durability or permanence under traffic, moisture fluctuations, and frost action in colder climates.

The concept of stabilization involves improvement of soil and aggregate material by one or more of the following procedures: drainage corrections, compaction, gradation changes, and use of additives.

The highway superintendent employs any combination of the various stabilization procedures to

1. Improve the bearing capacity of existing subgrade soil,
2. Modify the physical properties of an unsuitable or questionable base course material,
3. Obtain maximum performance from suitable base course material,
4. Reduce the total pavement thickness for a given traffic load, and
5. Provide a satisfactory wearing surface for secondary roads with low traffic volume.

The properties of calcium chloride that make it a particularly useful additive for stabilization are as follows:

1. It has an attraction for moisture;
2. It has low vapor pressure, which enables the chemical to resist evaporation;
3. Its solution has a high surface tension, providing the ability to bind aggregate particles together; and
4. Its solution has a strong moisture film; lubrication of the aggregate particles helps in compaction. The result is greater density through more effective compaction.

Calcium chloride's attraction for moisture and its low vapor pressure maintain uniform moisture content while the recycled material is graded, rolled, and cured. Maintaining the optimum moisture content in the base is the chemical's greatest contribution to the stability of the pulverized material.

Most full-depth reclamation of asphalt roads with calcium chloride is done on roadways in advanced stages of deterioration, with alligator cracks, rutting, frost heaves, potholes, and so on. The work consists of pulverizing the existing surface and blending the crushed surface with its gravel base to a desired depth, adding 0.75 gal of liquid calcium chloride per square yard, repulverizing, grading, rolling, adding 0.25 gal of liquid calcium chloride per square yard to prevent raveling, and letting it cure. Work has been done with Bomag MPH 100 recyclers, and Caterpillar R.R. 250 and Barber Greene RX-40 Dynaplane machines with savings in time, material, additives, and money.

CASE HISTORIES

Niskayuna, New York

In 1978 the highway department of Niskayuna, New York, discovered a solution for one of the town's most serious road maintenance problems. The Niskayuna highway department had always saved and reused old blacktop to create a new aggregate base and had been using calcium chloride for dust control for a number of years. It was a natural next step to combine the two operations, with impressive results.

General Chemical Corporation, 90 East Halsey Road, Parsippany, N.J. 07054.

Niskayuna's biggest budget item was for blacktop, but the budget did not keep pace with the price of blacktop. It was a losing situation until the calcium chloride full-depth reclamation solution. The procedure used was to scarify out blacktop and the gravel base and pass them through a crusher. This homogeneous mass of crushed asphalt and gravel was then redeposited over the subbase. After the homogeneous mass was laid out, 35 percent liquid calcium chloride was applied at the rate of 0.60 gal/yd^2. Three days later liquid calcium chloride was applied again in two passes at a rate of 0.25 gal/yd^2 during each pass. The road was then graded and rolled. Within a few days the road started to harden. A finished surface course of 1½ in. of plant mix topped the road.

In 1979 and 1980, the Niskayuna highway department reconstructed 1 mi each year with similar results. In 1981, 5 mi was reconstructed. By scarifying, combining the blacktop and the gravel base, crushing them, and then adding the liquid calcium chloride to bind the aggregate, Niskayuna was getting reconstructed roads at half the material cost and staying within their budget.

Colonie, New York

In 1983 the town of Colonie, New York, used a Bomag MPH 100 recycler to a depth of 12 in., 6 to 8 in. of which was asphalt that was pulverized with 4 in. of gravel base. On the second pass with the Bomag recycler, workmen were constantly having to stop the operation of the Bomag machine to adjust the amount of emulsion to add. Either too much was added to the recycled material, which made it bleed through, or not enough to bind the recycled aggregate. Because of the success that the neighboring town, Niskayuna, had had with liquid calcium chloride, a decision was made to try this additive.

After the asphalt road had been pulverized to a depth of 12 in., graded, and rolled, liquid calcium chloride was applied to the road through an Etnyre distributor at a rate from 0.60 to 1 gal/yd^2. Within a few days the road started to harden, and 1 month later it looked just like a paved road. Later that summer, the road was paved. The average daily traffic on that road is 1,500 to 2,000 vehicles per day.

East Greenbush, New York

Several towns in the area of Colonie, New York, experimented with liquid calcium chloride. In the town of East Greenbush, New York, liquid calcium chloride was substituted for an emulsion additive. There were assurances when the work began that liquid calcium chloride could do the job and would cost about 60 percent less than the emulsion. During the recycling, the road was kept open and watched carefully. The Director of Public Works was surprised that the road became so hard. After 4 weeks, a double seal of oil and stone was applied.

During the summer of 1983, because of the success of their test road, the town of East Greenbush, New York, reconstructed an additional 3 mi of roads with the Bomag MPH 100 recycler and liquid calcium chloride. Over the last several years the town has reconstructed 20 mi. Some of the roads were not surfaced, and to this day they have still not been surfaced, but remain smooth aggregate roads.

Sempronius, New York

For over 15 years the town of Sempronius, New York, experienced problems with a heavily traveled oil-and-stone road. Every year the town was faced with repairing potholes, alligator cracks, and ruts. The road surface always remained a problem because engineers were not convinced that they had a solid base along the entire length of the road. At first they thought of tearing up the whole road, but the cost was prohibitive. An alternative was to continue filling potholes and washed-away areas. Trueing and leveling would not cure the problem. They would cost more in the long run and the road would still be in bad shape. A cost of $30,000 was estimated to pave the road. This was compared with $18,000 to recycle the road with liquid calcium chloride, including a sealing of oil and stone.

The decision was made to try full-depth reclamation. This included pulverizing with a Barber Greene RX 40, grading, shaping, and rolling, plus two applications of 0.40 gal/yd^2 of 35 percent liquid calcium chloride. Done the old way, reconstruction would have required over 2 weeks of labor.

The Barber Greene RX 40 made passes starting at the edge of an 18-ft-wide road and traveling about 40 ft/min. On each pass the machine pulverized the road to a width of about 6 ft and to a depth of 6 in. The machine ground up the 1 in. of oil-and-stone surface and 5 in. of gravel base. The homogeneous mass of oil and stone, dirt, and gravel was then distributed evenly along the road. A grader reshaped the road, forming a crown with an elevation of 0.25 in./ft. At this point, an asphalt distributor truck applied liquid calcium chloride at a rate of 0.40 gal/yd^2. After the second application of the same amount, the road was then rolled.

It was the town's intention to seal the road with oil and stone; however, the road was so hard and was standing up to traffic so well that it was decided not to apply a wearing course. After a summer of heavy traffic by 10-wheel tractor-trailers, commuters, and service vehicles, plus heavy rains, the road remained hard and dust free. Motorists thought that the road was paved.

PROCEDURES

Stabilization and recycling of existing roadways are not new. There are different procedures, using different types of equipment, such as scarifiers, hammermills, mix pavers, and pulvimixers. The newer reclaimers, such as the Bomag MPH 100, Caterpillar 225, and Barber Greene RX 40, make it economically attractive for in situ full-depth reclamation roads.

Several factors need to be considered when the road is set up for full-depth reclamation. Mix design, which includes chunk sizes and material gradation as well as binder type and amount, must be determined. Laydown requirements must be determined. An economic analysis should be carried out to compare the cost and savings of this method of pavement rehabilitation with alternative pavement maintenance strategies. In every instance, when all the factors were considered, full-depth reclamation of an asphalt road with calcium chloride was tried. The results were successful.

Most in-place asphalt stabilization and recycling projects consist of a series of operations:

1. Ripping or scarification of the existing pavement layers and gravel base,
2. Reduction in size of the asphalt-treated aggregate particles and gravel base,
3. Mixing in of the new asphalt binder with treated aggregate particles and base material,
4. Spreading the recycled material, and
5. Compaction of the recycled material.

The factors in the recycling series of operations, which include the depth of the road to be recycled and the depth of the asphaltic material in the road, include the following:

1. A scarifier, a hammermill, and a grader are needed. A Bomag or a Caterpillar reclaiming machine can do this series of operations in one pass.
2. Mixing in of the new asphalt binder is the most critical operation in the recycling process. What binder? How much binder? How much asphaltic surface is going to blend with the new binder? How will the gravel absorb the binder? According to Scherocman (*1*):

> Any type of asphalt material—asphalt cement, foamed asphalt, cutback asphalt, asphalt emulsion or recycling agent—can be added through the recycler from the tank on an asphalt distributor. In recent years, emulsified asphalt has been the primary binding agent used in most cold in-place recycling projects. The primary decisions to be made during the mixing operation revolve around the type of asphalt binder to be added and the amount to be used. Again, depending on job conditions, one or more passes of the recycler may be required to properly distribute and mix the asphalt binder with the reclaimed material. Because of this multipass operation and because of the variability of this binder addition process, the uniformity of the binder distribution is sometimes poor.

3. Liquid calcium chloride can be mixed into the reclaimed material easily and with less margin for error than with asphalt emulsion. Liquid calcium chloride can be added through a distributor on the surface of the pulverized road material and then repulverized to the desired depth.
4. The spreading or grading of recycled material with calcium chloride is done in the conventional way with a grader.
5. Conventional compaction equipment—a static steel roller, a vibratory roller, or a rubber-tired roller—is used to provide the desired density to the cold-recycled mixture.
6. The amount of 35 percent liquid calcium chloride does not vary and there is less margin for error than with asphalt emulsion. For example, in a desired cut from 6 to 8 in. 0.75 gal/yd^2 is recommended after the first pass and 0.25 gal/yd^2 after rolling. For a depth of 4 to 6 in., the same amounts are recommended.

There are engineers who say that because of unknown factors of mix design that can significantly alter the level of performance of the full-depth reclaimed material, a wearing surface should always be placed over the recycled mixture. This is not the author's experience. Wearing courses can be single- or double-surface treatment, a layer of cold-mix asphalt, a layer of asphalt concrete, or just the full-depth reclaimed material that has just been treated with liquid calcium chloride. Twenty percent of the full-depth reclaimed asphalt roads with calcium chloride remain without a surface course.

There are certain recommendations in recycling with calcium chloride:

1. The asphaltic surface must always be blended with the gravel base course.
2. The gravel base course must be free of 4-in. bones and cobbles, large boulders, rocks, tree stumps, and so on.
3. There must be limitations as far as gradation is concerned with the reclaiming machine. Chunks of asphaltic material may be larger than 2 in. The percentage of −200 mesh is sometimes less than 3 percent. If the depth of cut is 4 in., fines must sometimes be added. It is extremely difficult to achieve 100 percent passing through a 1-in. screen, which is a typical specification for materials on a calcium chloride recycling project.

MATERIALS

The materials should be a mixture of bituminous concrete and existing gravel base course pulverized to conform to the following gradation:

Sieve Designation	*Percent by Weight Passing*
2 in.	100
1 in.	30–65
No. 200	3–12

Allowance must be made in the specifications for the inherent variability of the full-depth reclaimed material. In most projects the above specification is met after the second recycling run, mixing the calcium chloride and the recycled material to the desired depth.

CONCLUSION

Full-depth reclamation of asphalt roads with calcium chloride in lieu of traditional construction methods saves money, natural resources, energy, and time.

1. Money: With a reclaiming machine the cost of pulverization of the road runs from $1.00/yd^2 to $2.00/yd^2, depending on the depth of the cut. The cost of the 35 percent liquid calcium chloride averages $0.75/gal.
2. Natural resources: In situ materials such as asphalt surface courses are pulverized and mixed with the gravel base courses.
3. Energy: No oil used.
4. Time: The average time to reconstruct 1 mi is 2 days.

The concept of calcium chloride as an additive in reclamation has been in use since 1978. Benefits of this additive in reclamation are as follows:

- Uniform Moisture Control: The most important factor in obtaining maximum density in a well-graded mixture is the maintenance of the optimum moisture content. Because of its low vapor pressure, calcium chloride in solution resists evaporation, even in periods of low humidity and high temperature.

- Increased Density: Calcium chloride increases the surface tension. Moisture films of calcium chloride solutions are stronger than those of plain water. The treated aggregate attains a greater density than untreated similar materials.
- Less Compactive Effort Required: Less rolling is required. The accelerated compaction permits earlier completion of work.
- Less Binder Material Required: Because calcium chloride aids soil fines in maintaining a moisture film, it proves an adequate bond for the aggregate.
- Surface Uniformity: The ultimate aim is a smooth riding surface free from long transverse and longitudinal variations, which are detrimental to smooth riding and easy driving. Moisture retained in the road permits the base course to be carried as an open surface without excess wear and deterioration due to traffic for an indefinite period before being primed with bituminous materials.
- Controlled Curing for Increased Stability: The results show that calcium chloride used in the mix ensures a high structural stability by controlling the rate of drying in both the compaction and curing periods.
- Dust-Free Surface.
- Improved Bond: Calcium chloride is an aid to the absorption of bituminous materials. Priming materials are readily absorbed and there is no block of bituminous materials due to dust film.
- Adaptable to Stage Construction: Calcium chloride aids in keeping the aggregate in place. Because of budget problems, engineers have been inclined, especially in rural areas, to recommend building roads in stages to check the grade, drainage, and the selection of the surfaces.
- Extends the Road-Recycling Season: Because of the low freezing point of calcium chloride, recycling work can begin just after the frost is out of the soil and extend into late November.
- Frost Protection: Small percentages of calcium chloride are effective in reducing detrimental frost action. Work done by Floyd Slate of Purdue University concluded that calcium chloride, in a stabilized mixture, prevented detrimental frost heaving (*2*).
- No Threat to Environment: According to George Momberger, a senior engineer technician with the New York State Department of Environmental Conservation, if calcium chloride leaks into a stream, it will carbonate out and leave the water.
- Economy: The average price of 35 percent liquid calcium chloride is $0.75/gal furnished and applied. A total of 1 gal/yd^2 is recommended.

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Special Challenges of Defending Construction Claims Related to Low-Volume Roads

JULIA L. PERRY

The author relates the special challenges of defending construction claims related to low-volume recreational roads as based on her experiences as counsel for the Eastern Federal Lands Highway Division of FHWA since 1984. Specifically, the author addresses (a) the unique design of low-volume roads, including the limited right-of-way and the efforts to design to "fit the land" and to provide a more accommodating ride; (b) the special qualifications required of the contractors, who may have to work in a "tight," or environmentally sensitive, area; (c) the difficulties of administering a contract in low-volume areas owing to the isolated nature of many low-volume road areas; and (d) the lack or misapplication of standard references, which were developed primarily for high-volume roads, and the impact on the potential for claims and on the focuses, strategies, and litigation support necessary to defend them.

As counsel for the Eastern Federal Lands Highway Division (formerly Eastern Direct Federal Division) of FHWA since 1984, the author has learned that there are still pioneers in the business of building roads, and many of them are working with low-volume roads. The Interstate system in the United States has been documented and standardized to the point that its design, construction methods, and even contract administration have become among the basic references for those in the field of highway construction. But the large number of potential users (i.e., the anticipated daily traffic) requires that the Interstate system, and other high-volume roads, of necessity incorporate a "first class approach," including the best engineering, materials, and low-risk contract administration, because of the serious potential impacts of such choices. For example, a simple collision ("fender bender" in American slang) between two cars at 55 mph is a far more serious matter than one at 35 mph. Low-volume roads, conversely, offer the potential for innovation, forced in part by the accompanying restrictions of the use of low-budget engineering concepts, because most low-volume roads are not granted the higher budgets per mile of the Interstate and primary systems. Thus, because low-volume road projects must use constrained right-of-way, locally available materials, short-cut site exploration, and other potentially high-risk procedures in order to meet their limited budgets, they encourage engineering innovation. Although this is not to say that innovation is not possible in the construction of Interstate and other high-volume roads, a primary source of innovative techniques in the highway construction field in recent years has been the construction of low-volume roads.

This makes low-volume road construction an exciting field in which to be an engineer and a challenging field in which to be a lawyer, because construction claims frequently focus on those clauses, drawings, and so forth, that do not have a history of courtroom interpretation, that is, those that are by definition innovative. The special challenges of defending construction claims related to low-volume roads as based on the author's experience are addressed, specifically (a) the unique design of low-volume roads, (b) the special qualifications required of the contractors, (c) the difficulties of administering a contract in low-volume areas, and (d) the lack or misapplication of standard references and the impact on the potential for claims and the focuses, strategies, and litigation support necessary to defend them.

The author is not an engineer and does not have engineering training. Her viewpoint is of a lawyer who works daily with highway engineers in determining how to protect their work from the crippling impact of undue litigation. If engineers spend the majority of their time in court defending their work, they cannot carry out their assigned program. The description here of the engineering techniques that they employ is from that viewpoint and is not intended as an instructional treatise on engineering itself. Although this viewpoint may not at first appear valuable to an engineer, it should be noted that when engineers are involved in litigation regarding their work, the judge is more likely to have the author's background than theirs and even less experience in working with engineers. There is no court in the United States that hears only cases involving engineering concepts. Even the Boards of Contract Appeals, discussed later, limit their case law to anything involving contracts, including everything from general purchase to engineering administration. The result of this system is that engineering concepts must be explained to a nonengineer for a just decision to be issued. That is the lawyer's role and the role of the engineers with whom he or she works.

DESIGNING A LOW-VOLUME ROAD

Limited Right-of-Way

Unlike the Interstates, where sufficient right-of-way is acquired to allow a "total destruction" path for construction, low-volume roads are frequently designed to be sensitive to their environment, because of the nature of the land, the

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deliberate effort to save adjoining properties, or the lack of funding to purchase a wider right-of-way. This means that the design of a low-volume road must often accommodate a limited width of right-of-way rather than the right-of-way accommodating the design.

To Fit the Land

In addition to the limited right-of-way, the design may have to take into account the fit of the road to the land. This may mean putting in a curve to avoid a body of water or a sudden elevation of the land rather than making a visually severe cut. It may mean designing the road so that it has a minimum aesthetic impact on the surrounding area, through use of colored pavements or specially detailed facings, for example. It may mean designing the road so that it has a minimum environmental impact, such as through detailed drainage considerations in a wetlands area.

To Provide a More Accommodating Ride

Roads built with recreational considerations, such as those in state or national parks or forests, may be designed to make the ride more accommodating to the driver. This may mean softening or decreasing the degree of change throughout a curve. It may mean planning the roadway to provide a vista beyond the road. It may mean designing pull-outs and parking to make better use of the area. It may also mean including more directional or information signs in the total roadway package. But the same road may also be used by—and must accommodate—commuters (in the case of urban parkways, such as Rock Creek Parkway in Washington, D.C., which are both recreational and employed by local residents as part of the larger urban network), large recreational vehicles (RVs), tour buses, or even log trucks.

Impacts of Design Limitations on Litigation

For all of the reasons stated above, design and construction in accordance with the design must be more strictly controlled than in the design of high-volume roads. The law frequently examines the process involved in the interpretation of a design into construction with what is referred to as the "reasonable contractor" rule. In accordance with that rule, if a reasonable contractor would assume certain tolerances in the construction of a design, those tolerances will be accepted by the court. Nevertheless, what is reasonable for a mass-produced high-volume road with a wide right-of-way may not be reasonable for a site-specific designed low-volume road with special environmental, aesthetic, and rideability intentions. Therefore, it may be necessary for the lawyer handling the defense of a claim that addresses the feasibility and necessity of design constraints on a low-volume road to educate the judge in the special purposes of the particular road. This may be done by having someone testify as to why the road was designed in the manner that it was. Those preparing the contract for the construction or modification of the road must also make a decision, at that earlier stage, whether the specifications should emphasize that these additional considerations have been taken into account in the design. As someone who must later prepare the defense of such specifications, the author favors inclusion of a standard clause that identifies the particular type of road involved. With such a clause, a judge can see that a contractor's arguments based on what is reasonable for a high-volume road would be inappropriate for that particular type of low-volume road. Some examples of such clauses are as follows:

xxx.xx This project is being constructed in an environmentally sensitive area. The project was designed taking into consideration the potential impact on the area of the construction and the completed project from an environmental viewpoint. The contractor should be aware that specifications that are intended to control environmental impacts will be strictly controlled. This language is not intended to suggest that any other specification of the contract will not also be enforced.

xxx.xx This project is being constructed in a national park and is intended for use by the park visitors. The project was designed taking into consideration the potential aesthetic impact of the completed project. The contractor should be aware that specifications that are intended to control aesthetic impacts will be strictly controlled. This language is not intended to suggest that any other specification of the contract will not also be enforced.

xxx.xx This project is being constructed in a residential area. The project was designed taking into consideration the potential impact of the construction and the completed project on the residents. The contractor should be aware that specifications that are intended to control such impacts will be strictly controlled. This language is not intended to suggest that any other specification of the contract will not also be enforced.

xxx.xx This project is being constructed in an agricultural area. The project was designed taking into consideration the potential impact of the construction and the completed project on the surrounding farms. The contractor should be aware that specifications that are intended to control such impacts will be strictly controlled. This language is not intended to suggest that any other specification of the contract will not also be enforced.

One of the primary problems of construction law is that of interpreting what a uniquely written specification requires. It is not uncommon in claims litigation, in discussing the potential interpretation of one state's specification for which there is no state court case law on point, to refer to how another state's court has interpreted it. However, although comparing one state's specifications for high-volume roads with another state's specifications for high-volume roads may have some validity, comparing high-volume road specifications with low-volume road specifications is like comparing apples and oranges—it frequently does not work. In one specific instance involving the Eastern Federal Lands Highway Division, a contractor who was awarded a contract to build a road designed to accommodate restricted traffic in an extremely environmentally sensitive area tried to argue that the specifications should be interpretable by reference to standard state specifications. It became necessary during the course of the hearing for the contractors' side to present evidence that the aver-

age daily traffic on a nearby Interstate was nearly 1,000 times higher than the traffic for which this particular road had been designed. Therefore, standards developed to provide for an adequate road base for Interstate roads were simply not relatable to the specifications regarding subbase on the low-volume road. This sort of argument becomes necessary when a contractor is attempting to offer what he considers a reasonable interpretation of specifications through analogy and when the low-volume road's specifications are actually so unique as to not be analogous to any other standards or specifications available.

It places an additional burden on the defendant of a claim when the majority of precedent law on highway construction claims relates to high-volume roads. The defendant of a low-volume road claim must first discount the case law relating to high-volume roads as not being adequately analogous and then convince the board or court to develop new case law relating to low-volume roads. An example is the case law relating to the aesthetics of a roadway design. Of the less than two dozen cases addressing the issue of aesthetics and highway construction identified in a data base search conducted in March 1990, the majority addressed situations in which the parties arguing for an aesthetic viewpoint were organizations or individuals opposed to the construction of the highway, not the owners or administrators of such construction. (See Appendix A for examples of such cases.) It is hard with such a particular history to convince the courts that the government is indeed serious about aesthetic considerations relating to low-volume roads.

Another problem occurs when the special consideration is understood by those preparing the design but not articulated in the language of the specification. The owners or administrators of an aesthetically planned road must recognize that, to the courts, a change for aesthetic reasons is still a change. There is no open invitation in contract law to those making the substantial effort of preparing a uniquely designed road to "correct" their work as the construction proceeds. Although this "on-course correction" may appear to be the clearest solution from an engineering standpoint, it creates a substantial potential for claims. (See discussion on case law on point in Appendix B.)

In addressing claims involving design standards on low-volume roads, the attorney must rely more heavily on the engineering and technical assistance available. Because the prevalent standards are for high-volume roads and are therefore not analogous, the primary strategy in addressing a design-related claim is the use of expert witnesses. Frequently those experts are the designers themselves, but the use of outside experts may also prove valuable.

CONTRACTOR QUALIFICATIONS

The unique design considerations addressed above also affect the need that the contractor be qualified to perform the work. A contractor who has no experience in performing work in environmentally sensitive areas, for example, may be unable to properly assess the potential costs for performing such work. Not only may the techniques of performance vary, but the preliminary stages—including obtaining appropriate permits—may be far more time-consuming, and therefore costly, than he had realized. Similar considerations are present for contractors who are constructing or reconstructing roads in remote areas for the first time. Access to supplies, labor, equipment repair, and even the site may be factors that have to be taken into account in determining both costs and techniques. The potential impact of these considerations becomes all too clear as the contractor gets deeper into trouble. There are three major impacts: delay claims, differing site condition claims, and terminations for default.

Delay Claims

From the viewpoint of a claims defense attorney, it seems that contractors never believe that delays are their fault. Of course this is true because claims never get to the attorney unless the contractor believes the delay was not his fault. Among the excuses for delays are that (a) the contract was generally inadequate (no specific inadequacy being referenced), (b) the contract was too strictly enforced, (c) the government did not protect the contractor from the government's subcontractors, and (d) the government did not allow the contractor to decide when his own workers should be endangered. Obviously, none of these excuses is directly addressed in the standard contract clauses on delay.

Analysis of a delay claim should begin with a straight examination of the facts—what happened, what happened next, and so on. Whose fault was it and how much it really increased the costs and who is going to pay for it are all secondary issues to the issue of what happened. Thus the most valuable tool in defending a delay claim is the project records. It is preferable in such situations not that the project engineer say anything and understand what was going on, but that he write down everything he sees. The first rule in defending a delay claim is to simplify it to the basic facts, and the lawyer cannot do that if the stories conflict and there is no record to answer questions. Basic contract administration (discussed further below) thus becomes crucial.

The facts enable the reconstruction of whether the delay was the government's or the contractor's fault. There is an advantage in working with low-volume road projects at this stage in that the lawyer can walk the project without being run down by a tractor-semitrailer. The projects are generally limited overall—in length, in people involved, and in contract items addressed. A low-volume road attorney can get to know the project intimately, and that enhances his or her potential presentation in court. The attorney can also get to know the people involved, including the contractor's laborers, since the contractors who can submit a responsible low bid are frequently local and have a fairly stable work force. Three years after the project has been completed—not an unusual length of time in contract claims litigation—it is still possible to trace most of those who worked on the project. The biggest concern then becomes which of several to call as witnesses rather than who if anyone can be found, as might be the case with a high-volume road. Each of these factors allows a more detailed defense of a delay claim.

Differing Site Condition Claims

As mentioned earlier, because of the isolated nature and special design characteristics of low-volume roads, the contractor's understanding of the site may be quite limited. This results in a greater potential for differing site condition claims. If the contractor's ability to conduct a thorough site investigation is limited by the site, he is more likely to skew his bid. In the same light, the office responsible for the design of the job may have conducted a more limited geotechnical or other location review. Nevertheless, the design must be based—as in any project—on some assumptions, and sometimes those assumptions are inconsistent with the actual conditions that are discovered after work at the project site begins. When actual work at the site reveals that the representations made in the contract are incorrect, that is the basis for what is known as a Category I differing site condition. When the contract makes no representations but the contractor indicates that he was entitled to expect a differing situation than the one that he encountered, that is a Category II differing site condition. Category II is the real problem in defending low-volume road claims. The chance of such a claim increases with the decrease in the amount of actual experience—both in construction and in the specific geographic area—that the contractor has. Yet many contractors believe that low-volume road projects are easier to graduate to from such simpler construction efforts as commercial parking lots and residential developments, because of their limited size (such as length in miles) and lower overall cost. In fact, the truth is that the low-volume road is more likely than a high-volume road to present technical challenges because of its unique site, isolation, special design characteristics, and so on.

From a defense viewpoint, a Category II differing site condition on a low-volume road presents more of a challenge because there are less likely to be clearly available sources or obvious comparisons to other projects with which all parties (including the judge) may be familiar on which to base what a contractor should have reasonably expected in a low-volume road area. Although there is a substantial burden on the contractor to prove such a claim, it takes less to convince a judge who knows the area than either party. In those instances, preparing any kind of defense becomes an exercise in creativity. Again, this is an instance in which the assistance of an expert witness should be considered.

Termination for Default

When a contractor underbids a job—for any of the reasons referred to above—there is a greater likelihood that he will be terminated for default, either for financial difficulties or for failure to make progress for other reasons. The less experienced the contractor is, the more likely he is to mismanage a job and increase the costs of performance and, since construction payments are based on percent completion of work, the more likely he is to decrease his rate of earnings. Yet the less experienced the contractor is, the more likely he is to underbid the job. It is a vicious cycle, and when combined with the tendency of inexperienced contractors to think that low-volume road projects are easy starter projects, it creates a greater potential for a termination for default. This problem has been exacerbated by the recently increasing use by contractors of individual or private sureties to bond the job. Individual sureties are less likely than corporate sureties to understand the potential complexities of any road construction job. Thus, when the contractor is defaulted, the individual surety is less likely to be able to adequately take over the job and complete it. In addition, the subcontractors and suppliers may have more difficulty in collecting from individual sureties, and they are likely to turn their frustrations on the contract administrator's representative on the site. The author strongly recommends that this additional emotional burden on the project engineer be recognized by his or her supervisors. One option is to refer the calls of the unpaid subcontractors and suppliers to someone other than the project engineer to coordinate—perhaps his or her supervisor or the office's counsel.

CONTRACT ADMINISTRATION

The isolated nature of many low-volume road projects can cause difficulty in the administration of the project and affect or contribute to legal claims. The potential problems include limited communication with supervisors and technical support staff (including the attorney—many a claim can be stopped at the dispute stage with a proper understanding of the chances of winning in court), inadequate staffing flexibility, inadequate records caused by limited office support or flexibility, and personality disputes affected by isolation. A contract administrator on an isolated project cannot go down the hall to get some help. A contract administrator on a high-volume project, on the other hand, may have access to established regional offices, local United States or state's attorneys, or universities. This situation is beginning to improve marginally for the contract administrator on the low-volume road project with the increased use in project trailers of computers, modems, facsimile machines, and telephones capable of conference calling.

The attorney must recognize that limitations on the administration of an isolated project may create special problems. The supervision may have not met the highest standards at all times because qualified substitutes were unavailable, such as in situations involving extended sick leave of the project engineer. It may be necessary, for example, to have everyone who served as project engineer meet with the attorney to reconstruct what happened on a particular project. In the same light, record keeping may have been inconsistent or incomplete. And contacts made with a supervisor in another area may not have been fully communicated to the people in the field. None of these problems are insurmountable with regard to litigation if they are recognized while the defense is being prepared. The problem occurs when they do not come to light until the hearing and no response has been prepared to an allegation of apparent mismanagement by the contract administrator.

The individuals who staff these projects must be aggressive about asking for the help they need. They must also think well enough of their own understanding of the requirements of the contract and their authority to administer it to stand

firm when challenged by the contractor. Such strong egos often result in exacerbation of disputes by personality clashes. This problem is also increased by the social interactions that arise in isolation. A contract administrator on an isolated project may either associate more with the position of the contractor than his or her supervisors in another area (and allow a lesser performance than should have been required) or feel that he is the "only outpost of justice" in the area and refuse to consider reasonable requests for flexibility.

The attorney who must step into the area of dispute when it becomes a claim would do well to start by identifying any personality disputes going on. This identification may, at the worst extreme, result in a decision to limit the use of the project engineer as a witness because his emotionalism would interfere with his credibility as a professional. Other individuals may be able to perform adequately as witnesses after discussing the situation with the lawyer, their supervisors, or a neutral third party. The attorney must also consider how the trial or hearing will affect future relationships between the project engineer and local contractors. Testimony should be factual, emotionless, and considerate, even if the attorney must work with the witnesses to achieve this. A "once-burned" witness is valueless on future claims.

VARIANCES BETWEEN FEDERAL AND FEDERAL-AID OR STATE LAW

What Is the Law?

There exist significant differences in the law applicable to contracts administered by the federal government and those administered by the states with federal financial assistance. The major reason for this variance is a codification of law known as the Federal Acquisition Regulations, or FAR, which governs all federally administered contracts. FAR is a peculiar beast, since it was intended to cover all federally administered contracts, whether they are for the purchase of a major weapons system, the acquisition of a service contract to provide cleaning services for any Army base, the purchase of a single piece of specialized office equipment, the study of a government-related research issue, or the construction of a road. Thus FAR contains many restrictions on contracting, including providing notice of availability, restrictions on noncompetition, award approval, bid protest procedures, contractor bonding, and contract modification, that affect federally administered contracts but not federally assisted contracting. Although there are states that have copied portions of FAR into their own contract administration law, the majority of state law is not comparable. Thus the low-volume road contractor is likely to be required to work with both FAR and local state contract law. The systems that are most likely to vary, depending on the extent to which a particular state has chosen to adopt FAR procedures, are the areas of bid mistake and bid protest, contractor bonding, and contractor novation or termination. The author has been contacted by contractors attempting to build a case against the state on a bid mistake issue by arguing their interpretation of FAR provisions, and no amount of pointing out that FAR is simply not applicable will dissuade them (nevertheless, the author always shares such contacts with the appropriate regional counsel for FHWA). Similarly, contractors will come before the boards and special courts attempting to argue that FAR has no greater weight than local state law for a project built entirely on federal land. Although there is no way to avoid such confusion on the part of contractors, assuming that the references in the contract are clear, the attorney specializing in low-volume road contract litigation should be aware of which jurisdiction of law governs.

What Kind of Court Is This?

The other unique aspect of federally administered contracting is the litigation system that has been established for it. Unlike in the states, claims are not filed in the principal federal or district court. Instead, federal law provides for a two-branched system to be used at the option of the contractor. After presenting a claim to the contracting officer and having it denied, the contractor can file an appeal of that denial either with the United States Claims Court or with the appropriate board of contract appeals. The Claims Court is a formal court, at a level equivalent with the district court, and appeal from it goes to the Court of Appeals, just as for the district courts. The Board of Contract Appeals, on the other hand, is an administrative court established in most of the executive departments. (Departments that have few contract claims, such as the Department of Justice, share the board of another department.) Generally, one judge presides at the hearing—the equivalent of the court trial—but the decision must be concurred in by the other members of the board. Procedures before the boards follow standard Rules of Federal Civil Procedure but are permitted some flexibility, depending upon the determination of the presiding judge. Thus, timetables for discovery and filings can be shortened by the judge in order to expedite the scheduling of the hearing itself. It is not unusual for a trial to be scheduled over a year after the initial filing with the Claims Court, whereas a hearing may be scheduled within 4 to 6 months, or, with an even more expedited procedure allowed for smaller claims, within 60 days. The documents submitted and the number of witnesses called by either party may also be restricted by the presiding judge.

Because of their structure, the boards are semiindependent. Appeal can be taken from them to the Court of Appeals, but this is unlikely. Thus the boards create their own case law on new issues. A board may or may not consider the position taken by their fellow boards, or even the position taken by the Claims Court. An example of this is the issue of whether a contractor could appeal a termination for default when he suffered no direct monetary costs. For years, one board held that he could, whereas the other boards and the Claims Court held that he could not. The issue was finally decided by the Court of Appeals in favor of the contractor. This peculiarity means that an attorney researching the relevant law must look first to whether the issue has been addressed by the Court of Appeals (which is unlikely), then to whether the Claims Court has discussed it (only occasionally true, because contractors tend to prefer the boards, which provide a faster decision), then to whether any of the boards has addressed it (more likely), and then to whether the particular Board that will hear the appeal has addressed it and has or has not agreed with the other boards or the Claims Court. The legal argu-

ments (or brief) must then address not just what the precedent law is, but which forum issued it and how persuasive that forum is to the one hearing the appeal. Contract law as it is being developed by the courts and boards thus is more closely analogous to a multibranched tree than to a simple linear progression from lower court to upper.

AVOIDING OR CONTROLLING LITIGATION

The primary issue in litigation involving low-volume roads is the same as that for high-volume roads: how can litigation be avoided? Litigation cannot be entirely avoided except by the unlimited payment of all claims presented. It can, however, be limited or controlled by the following efforts.

First, the contract should be written to anticipate potential problems. If a particular effort is required to comply with a specification, that should be noted. Perhaps that will educate—if not deter—the less experienced contractor. Special caution should be taken in the preparation of specifications that are intended to produce a subjective result, such as aesthetics. Courts will not enforce subjectivity and will treat such a clause as ambiguous. Thus, an aesthetic standard must be translated into an objective requirement.

Second, the contract should be enforced as written or else modified accordingly. The contract administrator's only defense is the contract itself. If the contract was not enforced, there is no other authority for the contractor's actions.

Third, in administering a contract, warning signs—such as a contractor's financial difficulty or failure to make progress—should be watched for and addressed promptly. An experienced contract administrator may realize that an inexperienced contractor is getting into trouble before the contractor does. Problems addressed at an early stage may keep the matter out of court. At worst, they put the government in the position of saying in court that everything possible was done to avoid or limit the impact of the problem.

Fourth, problems on the project should not become personal problems. The contract administrator has the right to require the contractor to do whatever is in the contract, or to require the contractor to perform in accordance with an appropriate modification, or to delay the contractor for a valid purpose. The contractor is entitled to be paid for his work, including the changes, or for any delay to performance. Neither is a personal issue. Personality only makes the legal arguments unclear.

Fifth, meticulous records of what occurred on the project must be kept. This is especially true if litigation appears imminent. Because not all litigation can be anticipated, it is better to apply this rule to all projects. Although recordkeeping is a major cut into an engineer's time (a recent Freedom of Information Act request to the author's division for all of the documents related to a specific project resulted in the identification of over 6,000 separate documents), good records can win cases.

Sixth, the engineer should work with the lawyer to educate him as to what the engineer was doing or trying to do on the project. The best legal cases in highway construction law are clear presentations of engineering positions.

Finally, the courts are impressed by the professionalism of engineers. The judges recognize their own limitations in understanding the specific considerations of highway engineering, and they will give great credence to an engineer-witness who is professional in demeanor, tone, and discussion. Therefore, an engineer witness should present himself with pride and speak objectively but with authority with respect to his areas of expertise. For purposes of this case and this judge, he is the noted expert in his field.

APPENDIX A: CASE LAW REGARDING AESTHETICS VERSUS ROAD CONSTRUCTION

Coalition Against a Raised Expressway v. Dole, U.S. Court of Appeals, 11th Circuit, Jan. 13, 1988.
This case held that noise impacts of a proposed raised highway could constitute "constructive use" of an adjacent park.

The County of Bergen v. Dole, 620 F. Supp. 1009 (1985).
In this case, a citizens' group opposed the construction of a segment that would complete Interstate 287 in New Jersey because of severe aesthetic impacts, despite the fact that studies of the proposed alignment included studies of the aesthetics from both the potential users' and the nonusers' viewpoints.

San Antonio Conservation Society Members v. Texas Highway Department, 496 F.2d 1017 (1974).
An earlier case decided that a road was still subject to environmental review despite the fact that the state had withdrawn its request for federal funds to build it. That decision was overturned in the cited case.

Indian Lookout Alliance v. Volpe, 345 F. Supp. 1167 (D.La. 1972).
In this case, the plaintiffs argued that separate impact statements should be prepared for segments of a proposed 1,900-mi roadway. The court held that it was too early to declare that the proposed segments were federal action, since only general corridors, and not specific routes, had been designated.

Citizens to Preserve Overton Park v. Volpe, 91 S.Ct. 814 (1971).
This is probably the best-known case in the field of environmental law versus highway construction. The local organization sought to stop the construction of a road that would take a portion of parkland.

Miller v. United States Department of Transportation, 710 F.2d 656 (1983).
This case may serve as an example of a contrary viewpoint. In this case, plaintiffs, a husband and wife who were injured when their car slid off of the road in icy conditions, charged that the Department of Transportation had acted improperly in approving plans to construct a portion of an Interstate in a mountainous area without guardrail, despite the fact that Title 23, U.S. Code, Section 109(h)(2), directs the consideration of "aesthetic values" in approving the design of any project. Thus aesthetics may be used as a tool by the plaintiffs in the environmental cases, but may not be an adequate defense in a tort claim.

APPENDIX B: CASE LAW INTERPRETING AESTHETIC SPECIFICATIONS

Appeal of Wayne L. Grist, Inc., ENG BCA No. 5503, Corps of Engineers Board of Contract Appeals, 90-2 BCA (CCH) P22,915, May 10, 1990.

The contract that was the focus of this litigation required the contractor to take steps to prevent "environmental pollution," which was defined in part as "the presence of chemical, physical, or biological elements" that "degrade the utility of the environment for aesthetic or recreational purposes." The board allowed this provision to be read with another provision to limit areas in which the underbrush that had to be cleared could be burned rather than excavated. The provision would not have been enforceable by itself, however, according to the interpretation of the board. The other provision held to be necessary to effect the "aesthetic" provision required that burning limits be set. Thus work that is otherwise required to be limited can be required to be limited for aesthetic reasons as well.

Appeal of Reynolds Construction, Inc., ASBCA No. 32047, Armed Services Board of Contract Appeals, 89-3 BCA (CCH) P22,126, June 28, 1989.

In this case, the government directed additional work (the application of a slurry seal to a ravelling road surface that appeared open-graded) as an "esthetic solution" for the appearance of the work. The contract did not contain a specification that addressed aesthetic considerations. And the board concluded that the open-graded appearance was the result of the aggregate size that the government had directed the contractor to use. Therefore, the board held that the government could not direct such a change for aesthetic reasons without paying for it. On the other hand, to the extent that the addition of the slurry seal corrected the ravelling—which the board held to be a workmanship defect—the government was entitled to direct the additional work without creating any entitlement in the contractor.

Appeal of J. D. Abrams, ENG BCA No. 4332, Corps of Engineers Board of Contract Appeals, 89-1 BCA (CCH) P21,379.

In this case, the government directed the contractor to add additional rock spalls to the wall of a dam in order to improve its appearance from a distance. The government argued that the method chosen by contractor to place the rock spalls created an unsatisfactory appearance, but the board held that the government had not specified how the work was to be performed nor how it should look when finished, and therefore had no basis to order the additional rock without creating an entitlement in the contractor. "[T]he Board is of the opinion that the visual criteria relating to the rock spalls which was imposed by the Government amounted to a constructive change for which Appellant is entitled to an equitable adjustment."

Appeal of Snider Lumber Products Co., AGBCA No. 78-171-5, Department of Agriculture Board of Contract Appeals, 81-2 BCA (CCH) P15,218, July 10, 1981.

In this case, the board upheld an amount set in the contract by the Forest Service for damage to or the cutting of trees as an impact on an aesthetic consideration. It is unlikely, however, that the board would have allowed such a deduction or damage charge if it had not been specified in the contract. In other words, the time to anticipate aesthetic damage is in the contract.

Gifford-Hill & Company, Inc., v. Federal Trade Commission, et al., U.S. District Court for the District of Columbia, Civil Action No. 74-1265, 389 F. Supp 167, November 13, 1974.

"All agencies which undertake activities relating to environmental values, particularly those values relating to amenities and aesthetic considerations, are authorized and directed to make efforts to develop methods and procedures to incorporate those values in official planning and decisionmaking." Dicta (i.e., a quote that has no value except to show the viewpoint of the court). Aesthetic considerations are "motherhood and apple pie," but they are not law.

Flood Disaster Rehabilitation, Charnawati, Nepal: A Case Study

J. KRAEHENBUEHL, W. OSTERWALDER, AND A. WAGNER

After a brief mention of the principles of design applied to the construction of the Lamosangu-Jiri Road in the central region of Nepal, the authors describe the effort undertaken to rehabilitate a road section heavily affected by a flood in the early monsoon season of 1987. Innovative techniques for the construction and maintenance of roads in the Himalayan region, such as anchoring, drilled subsurface drains, and flexible river protection, were introduced. In view of design options for low-volume roads in hilly terrain, practical suggestions such as low initial investment and event-related rehabilitation are discussed.

Extreme difficulty of access is one of the dominant characteristics of mountain communities and is a formidable constraint to the effective implementation of essential programs of rural development. This is the case throughout the hills and high mountain districts of Nepal. For this country, situated between India and China and dominated by the Himalayan range, road transportation is vital for the development of industries and commercial agriculture because other types of transportation (railway, ropeway, or air) are only complementary to road travel and do not contribute substantially to the transport of goods or passengers. The present road network in Nepal consists of approximately 7000 km, resulting in a density of 1 km per 21 km^2 (*1*). Most of these are gravel or dirt roads, and a large number of them can only be used in fair weather. Construction of all-weather roads through the mountains to improve access to remote areas is extremely costly and technologically challenging, particularly on steep and unstable slopes or across mountain rivers. Most road failures occur during the monsoon season (July to September) because of floods or slides, which sometimes are triggered by earthquakes.

LOCATION OF THE ROAD AND PROJECT AREA

The road considered here is situated in the central region of Nepal. It starts from Lamosangu at the Arniko Highway (linking Nepal to China) and after 110 km, reaches Jiri, the district headquarters and starting point for porter and trekking services to central and eastern Nepal (Figure 1). The road was constructed between 1975 and 1985 with contributions from the Swiss Development Cooperation.

The area requiring extensive stabilization work, described later, is located approximately midway (Km 45), at an elevation of 1800 m where the road crosses the Charnawati River.

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ROAD STANDARDS

The Lamosangu-Jiri Road was constructed to a single-lane standard with an overall width of 5.4 to 6.1 m and bituminous surfacing (*2,3*). The design speed was 30 km/hr, and the maximum longitudinal gradient was limited to 12 percent. It was designed considering local conditions as much as possible, and the following guidelines for implementation were followed:

- Close cooperation with local administration,
- Consideration of ecological constraints in order to prevent adverse environmental impacts and minimize maintenance costs,
- Use of labor-intensive methods to create job opportunities for the local population and maximize the financial input into the region,
- Training of local people, and
- Close cooperation with other development projects in the region.

In summary, the road was constructed with a low-cost approach and labor-intensive methods. The use of equipment was reduced to the lowest level that ensured acceptable quality and rate of progress. This put a considerable constraint on the applicable engineering methods to cope with the frequent problems of floods and slides. All main structures were executed in gabions. Despite the advantages of their flexibility and excellent drainage capabilities, the reasonable height of retaining walls or dams constructed with gabions was, in general, limited to approximately 10 m. In addition, gabion wires exposed to tractive forces due to soil movement or to abrasion in rivers represent problems for maintenance.

Although the estimated (and actual) traffic was rather low, the road base was designed with a bituminous surface course (either a 5-cm penetration macadam or a 3.5-cm premixed asphalt layer using bitumen emulsion) on a width of 2.9 m. This solution was chosen mainly because of the steep gradients of the road in the hilly region and to improve drainage of the road body during monsoon season. The somewhat higher cost of a blacktopped road is more than balanced by easier maintenance of the road surface.

TRAFFIC

The Lamosangu-Jiri Road is a Class II feeder road according to Nepal Road Standards (*4*). Traffic forecasts used for road design in 1975 were of the order of 10 trucks per day in each direction, with a 5 percent increase per year. Observations

FIGURE 1 Location map with main road network of Nepal.

during the construction phase of the Charnawati Rehabilitation Project showed the following average traffic: 7 buses per day in each direction and 10 trucks per day in each direction. Maximum axle loads of trucks were estimated to be 13 tons, according to an axle survey conducted on Nepalese roads in 1983 (5).

FLOOD DISASTER AND CONSECUTIVE SLIDES

The Charnawati River has a gradient of 12 percent as it crosses the road. Gradients up to 25 percent can be observed further downstream. Its catchment upstream of the road crossing covers an area of 13 km^2 and is almost semicircular in shape. It is confined by very steep cliffs with a difference in elevation of approximately 1000 m. This morphology is responsible for the microclimate in this area, provoking very heavy monsoon rains in a limited small area. Rainfall and discharge measurements at tributaries of the Charnawati River showed that rainfall intensities vary considerably even within 1 km. The heavy rains often trigger mud and debris flows in the tributaries, disturbing the main flow of the Charnawati.

During the night on June 30, 1987, a heavy flood of the Charnawati River destroyed the original 10-m-span concrete bridge on the road and triggered slides all along its banks, particularly disastrous to the road for a length of approximately 1 km on the left-bank bridge approach. The river partially created a new bed in a process of erosion and sedimentation. Riverbed erosion to a depth of 5 m (locally even 10 m) and recent local sediment deposits up to a thickness of 3 m could be observed after the flood.

Hydrological estimates based on flood traces revealed that the discharge during the flood event was approximately 150 m^3/sec. A hydrological study using regression analysis and run-off formulas concluded in the following maximum discharge estimates: Q_2 = 40 m^3/sec, Q_{10} = 95 m^3/sec, Q_{25} = 120 m^3/sec, Q_{50} = 140 m^3/sec, and Q_{100} = 160 m^3/sec.

PROJECT HISTORY

At the end of the monsoon season in 1987, studies for the rehabilitation of the Charnawati crossing were undertaken and implementation of the Charnawati Rehabilitation Project (Phase I) started in January 1988. Due to time constraints, construction had to be limited to a road stretch of 300 m on both sides of the river crossing.

Simultaneously with the implementation of the first phase of the project, realignment alternatives were studied to bypass the critical section. It was found that realignments were either too long and thus too costly or passed through highly unstable areas. Thus the problems would have been shifted rather than avoided. A conclusion was therefore reached to maintain the alignment by trying to stabilize the slides at and below the road.

The target of keeping the road open during monsoon season in 1988 was achieved. However, four slides triggered by the 1987 flood, which were 100 m below the road, developed with unexpected speed during monsoon season 1988 and two of them reached the road by the end of it. This development called for substantial additional stabilization measures to save the road and the stabilizing structures, which constituted the primary investment. A second phase of the project was thus implemented from November 1988 to June 1989.

At the time of this writing, stabilization work was still in progress (Phase III) and is expected to be complete by the end of 1991.

Figure 2 shows an overview of the project area with the Charnawati crossing in the foreground.

GEOLOGICAL AND HYDROGEOLOGICAL CONDITIONS

The Charnawati area is located in the lesser Himalaya, in the so-called midhills that are incised by many rivers and are continuously reshaped by active erosion processes.

The whole region between Km 42 and 47 of the road has been evaluated by hazard mapping. The technique of soil hazard mapping has been developed based on the principles of rock and debris slide risk mapping (6,7) adapted to slide hazards in soil material. By this method a weighting factor is attributed to all relevant parameters governing the soil stability, such as

- Slope angle and type of soil (residual, colluvial, alluvial, or glacial),
- Hydrological and hydrogeological condition (perennial-seasonal streams, springs, water table),
- Hydrodynamic condition (floods, glacial lake outburst, slide dam breakthrough),
- Past and present slide and erosion activity,
- Seismic activity and presence of faults, and
- Land use (vegetation, forest, irrigation).

The combination of all parameters results in a soil hazard map (Figure 3) showing areas of low, moderate, and high instability. This map is used to design possible realignment alternatives and to define the locations where further detailed investigations are necessary. The map clearly shows the high-hazard area on the left bank of the Charnawati River (Km 45 to 46 and Km 47).

In this area the rock is predominantly gneissic and covered by a thick soil layer at most locations. The top soil layers generally consist of colluvium with a silty-sandy matrix and gneissic boulders (2 to 5 percent). Slopes are rather steep with typical slope angles between 30 and 35 degrees.

Detailed geophysical investigations, exploratory drilling, piezometers, and soil tests revealed the following subsurface conditions:

- The internal angle of friction of the noncohesive colluvial matrix is 33 degrees (consolidated undrained triaxial shear tests with measurement of pore-water pressure), classified as SM-ML with very low plasticity;
- The thickness of the soil cover above the bedrock in the critical (sliding) areas is greater than 20 m (resistivity measurements);
- The water table is situated at a depth between 3 and 9 m during dry season and is partially rising to the surface during monsoon;
- Seismic refraction tests showed a distinct change in wave velocity at a depth of between 3 and 7 m; this was attributed to the boundary between upper loose (sliding) and lower more compacted soil material.

FIGURE 2 Aerial photograph of project area, looking south; Charnawati crossing in the foreground.

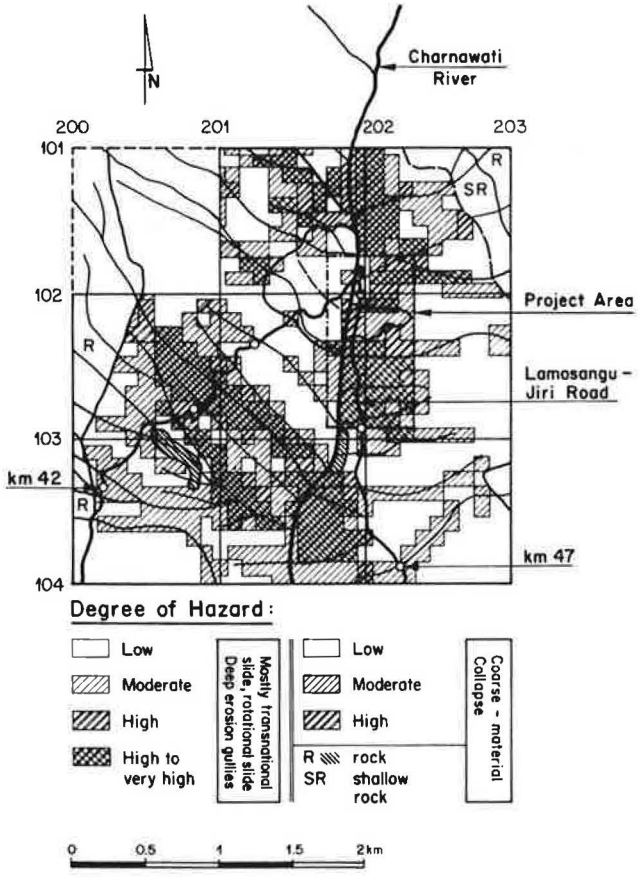

FIGURE 3 Charnawati Valley soil hazard map.

The latter investigations and stability computations (method of Janbu and Bishop), confirmed later by extensometers (see the next section), revealed that the slip surfaces controlling the slides are at a depth of between 3 and 7 m.

REHABILITATION WORK

Construction work during the project can readily be divided into four different groups:

1. River control work to stabilize the toe of the slides,
2. Drainage in and above the slides,
3. Slide stabilization work, and
4. Road rehabilitation.

Figure 4 shows a plan of the project area indicating the extent of the slides and the main structures.

River Control Work

The Charnawati River bed was stabilized and partially lifted for a length of 450 m by means of nine check dams, constructed using gabions with all exposed parts (crest, etc.) covered with concrete to avoid wire abrasion problems. The geometry and gradient of the riverbed called for a dam width of 13 to 23 m and a height of 4 to 6 m. The horizontal spacing of dams was between 30 and 50 m. Since all excavation work in the riverbed was undertaken manually, the foundation depth was limited to 1.0 to 1.5 m. To avoid scouring problems downstream of the dams, the aprons were reinforced by concrete slabs with a cover of stone slabs. Generally, the river banks between the dams were protected by gabion guide walls, thus buttressing the toe of the endangered slopes. Figure 5 shows the layout of the check dams in the vicinity of the river crossing.

Stabilizing the river bed with check dams represents a classical technique, although the size of the dams constructed can be considered substantial for Nepalese conditions. Since the series of check dams is retaining most of the bed load of the river, scouring will be pronounced at the last downstream dam. Construction of a stilling basin with controlled dissipation of energy, in which excavations to a depth of more than 5 m would have been necessary, was found not to be feasible because of the high gradient and the labor-intensive construction methods involved. Two alternatives were discussed to solve the problem: (a) continuation of the series of check dams downstream until a flatter area is reached, necessitating the construction of up to 40 more check dams of similar shape, and (b) reinforcement of the riverbed by concrete blocks (so-called flexible river protection). The latter alternative was found to be more attractive and less costly because protection work could be confined to the critical area downstream of the check dams.

The principle of flexible river protection is to make use of the existing boulders in the riverbed and to selectively reinforce them with concrete blocks. The concrete elements are shaped in a three-dimensional H-form similar to the elements used for breakwater protection. This allows better interlocking of the elements and still permits the low water to flow between them.

The initial layout of the concrete elements is of great importance and was extensively tested in hydraulic models. Most of the elements were placed at the riverside and on the banks (to act as stabilizers if the bank is undercut). In addition, the

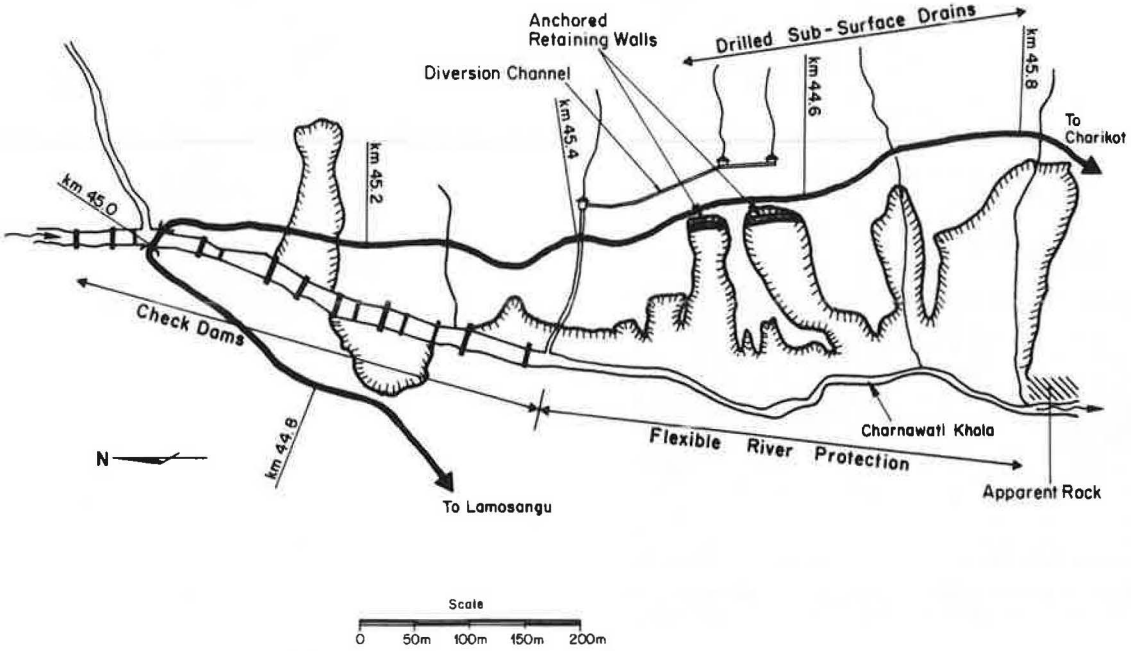

FIGURE 4 Charnawati Project: overall plan view. Only the main substantial structures are presented; drainage and biotechnical measures in the slides are not shown.

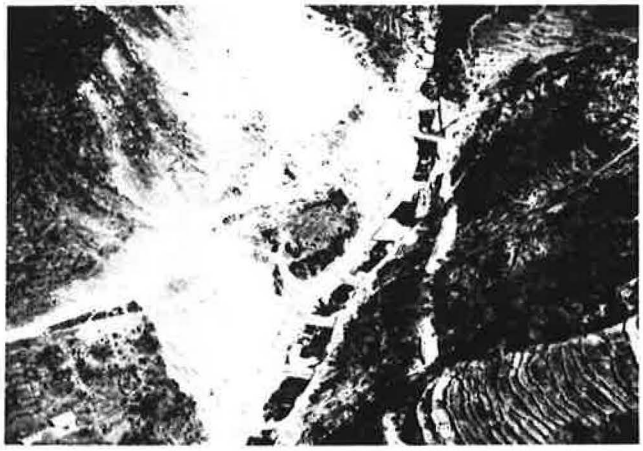

FIGURE 5 Aerial photograph of check dams and road crossing. Note the slide on the right-hand bank where the alignment had to be shifted to the mountainside. The cleared area to the left of the bridge was used for quarrying.

accumulation of concrete elements forming a sort of spur or sill proved to be efficient. On the average, the Charnawati River bed has a 90 percent fraction of boulders of 1 m in diameter, although some rare big blocks of up to 10 m in diameter can be observed. In view of the above parameters, the height and width of the concrete elements was chosen as 2.5 m, resulting in a weight of 10.5 tons per element. Upon completion of the project, 700 elements will have been placed in the riverbed over a length of 500 m.

Considerable effort was invested in construction planning. The concrete elements were cast in situ (in the riverbed) using steel formwork composed of manually portable pieces. Concrete from the batching plant was transported by cable crane to the river site. This technique allowed an overall construction progress of three elements per day.

Drainage

As is often the case, water (below and at the surface) was the key factor for most slides at the Charnawati banks. It was

FIGURE 6 Typical causeway for smaller rivulets.

FIGURE 7 Stabilized slide at Charnawati left bank, Km 45.1. Two main drains crossing the road are supported by the gabion guidewall at the river.

observed that the silty soil flowed like mud once it was completely saturated and the protective vegetation cover was destroyed. Access to these areas was impossible during monsoon time.

In principle, an attempt was made to control all surface water in the sliding area. A typical causeway with gabion protection, as used for cross drainage before the slides at Km 45.4 to 45.6, is shown in Figure 6. Surface water of two rivulets was contained above the road and diverted to a rivulet in a more stable area. All slide areas were drained to a depth of 2 m by gabion drains located in topographical depressions and aligned in the direction of the slope gradient, thus evacuating water by the most direct route. Tributary drains at intervals of 15 m were designed to collect water and guide it to the gabion drains. A successful application of this system can be seen in Figure 7, showing a stabilized slide with two main gabion drains crossing the road.

Stability computations (see previous section) showed that at the steep slopes between Km 45.4 and 46.0, drainage to a depth of 2 to 3 m is not sufficient to achieve reasonable safety factors, even in combination with retaining structures. It was therefore decided to implement horizontally drilled subsurface drains, which allow drainage to much greater depths. Although application of this technique, which is new in Nepal, necessitated a considerable investment in drilling equipment

FIGURE 8 General view of slides, Km 45.4 to 46.0. Drainage and biotechnical measures can be seen in the upper parts of the slides.

and operator training, the system proved to be the most effective in raising the safety factor to an acceptable level. A factor of 1.2 to 1.3 was considered reasonable under local conditions, because the achievement of higher safety factors would have involved prohibitive costs. The drilled subsurface drains are 25 to 35 m long to reach a depth of 5 to 6 m. They are arranged in fans so that up to 10 holes can be drilled from one spot, thus keeping the water table below 4 m even during monsoon time. The drains consist of perforated pipes (63-mm diameter) made of polyethylene and sheathed with geotextile.

Stabilization Work

Figure 8 is a general view showing the status of the upper parts of the four slides from Km 45.4 to 46.0 in February 1990. It shows that parts of the slides could be stabilized but that further work is necessary. At the upper edge of the two main slides in the foreground, anchored retaining walls were constructed just below the road to prevent further upward development of the slides and to retain the road. The layout of the walls and drainage efforts are shown in Figure 9.

Vertical series of four earth anchors were arranged at intervals of 4.3 m. The anchors consist of steel rods St 500/600 with a diameter of 32 mm and a length of 25 m (upper two anchors) and 20 m (lower two anchors). Anchor forces are built up by friction and dilatation effects between the grouted anchor hole and the soil. In the stability computations the permissible anchor forces were 250 kN per anchor. Pullout tests showed that values of 400 kN could be reached if boulders were hit during drilling, thus expanding the anchor root.

Since such anchoring techniques have not been applied in Nepal before, care was taken to monitor the behavior of the structures by means of extensometers, measuring deformations in the soil. The anchors are not prestressed; consequently the full anchor forces are built up only after deformations of approximately 1.5 cm. In order to have a structure compatible with these deformation requirements and to save some (expensive) concrete work, the wall was executed as a gabion wall with intermittent concrete blocks anchored back.

To complement the structures, all bare surfaces of the slides were treated with biotechnical measures to avoid water infiltration at the surface and to stabilize the slide surface. The techniques applied were

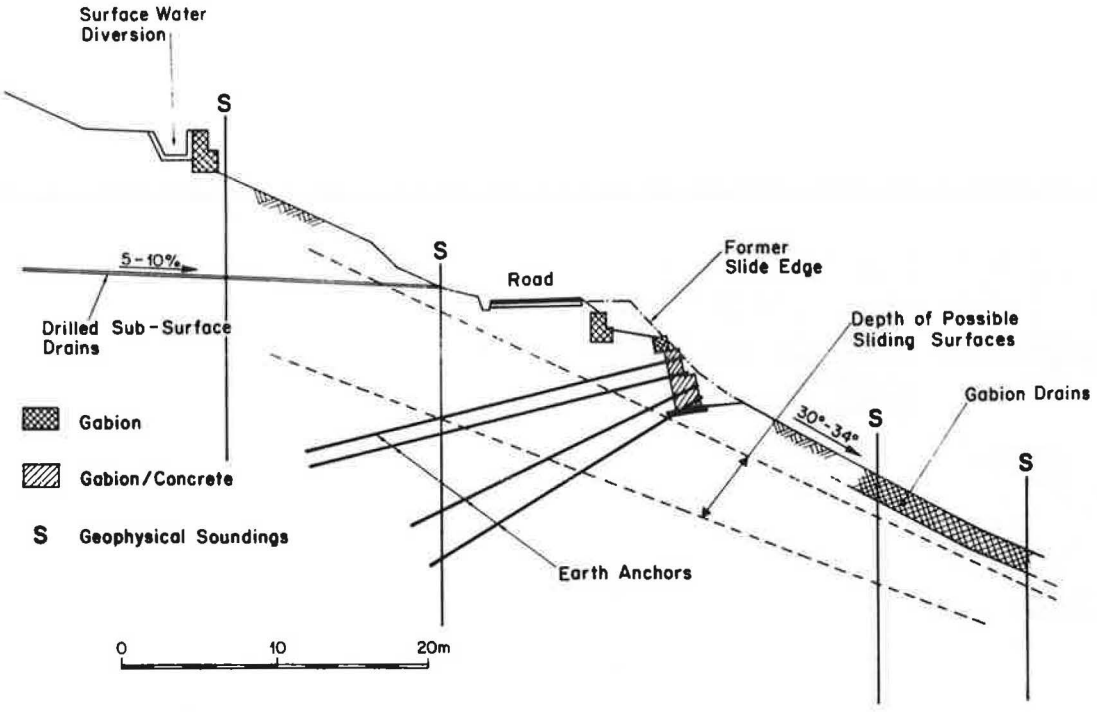

FIGURE 9 Cross section of slide stabilization measures at Km 45.4.

- Terracing of all slopes,
- Mulching of surface with branches of bushes that sprout during rainy season fixed with wire nets at steep slopes,
- Reinforcement of the top soil layer with stone arches (dry masonry layers arranged in arches to a depth of approximately 30 cm) and hedge brush layers,
- Sowing of grass, and
- Planting of locally growing deep-rooting trees

For these measures a nursery was maintained throughout the construction period. Successful applications of these techniques which are also aesthetically satisfying, are shown in Figure 7 and 8 (upper parts of these views).

Road Rehabilitation

Approximately 600 m of the Lamosangu-Jiri Road had to be reconstructed and the centerline needed to be shifted to the mountainside above the slides. On the basis of the traffic assumptions outlined earlier, the pavement was designed with 25 cm of untreated crushed aggregates as the subbase and 15 cm of water-bound macadam as the base course. An additional blacktop (penetration macadam) will be applied after completion of construction work.

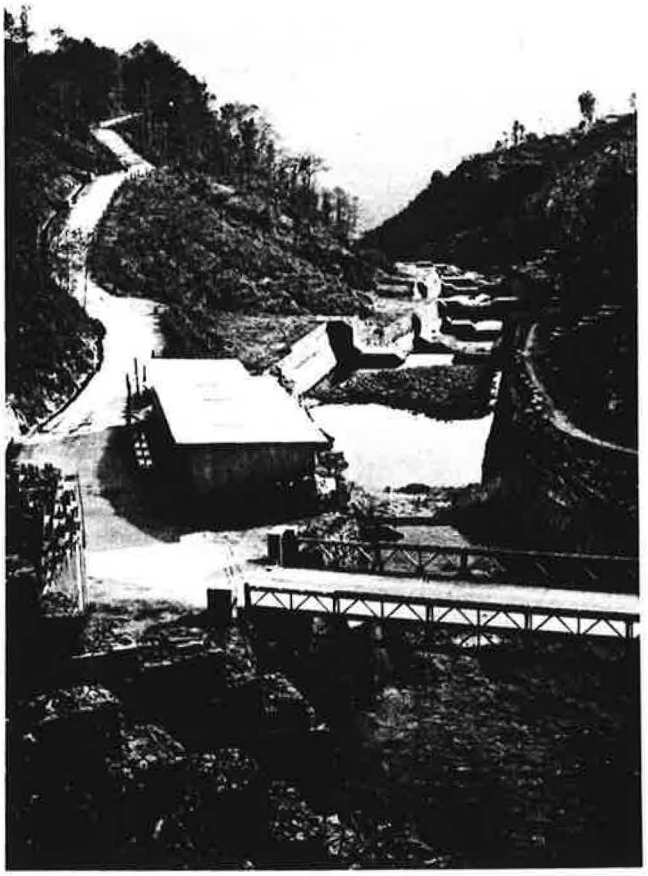

FIGURE 10 New Charnawati bridge crossing. The store in the foreground is used for current construction work.

Soon after the disaster the 10-m-span concrete bridge that was washed away was replaced by a Bailey bridge of 21.3-m span, which can be erected in short time (Figure 10). This bridge was initially meant as a semipermanent solution, but is likely to become permanent.

Summary of Rehabilitation Work and Cost

In general, structures for stabilization of the area were restricted to the minimum necessary to save the road. Prime importance was given to drainage, which resulted in the highest stability improvement at lowest costs. However, retaining walls to complement the drainage system and the stabilization of the riverbed were necessary to achieve long-term stability. Despite the rather high investment (see below), the safety factors achieved are not substantially above 1.0 and many parts of the slides, for example, the lower parts of the slides between Km 45.4 and 46.0, have been treated only minimally. Part of the stabilization work was left to nature in the sense that the sliding process was allowed to continue in areas less critical for the road until a new equilibrium was found. Only then were supporting measures, for example, reinforcing the toe of the slide, undertaken.

It must be emphasized that, despite the involvement of special equipment for the innovative construction techniques described above, the main structures were built with labor-intensive methods. For example, all earthwork was executed manually. Because of the time constraints implied by the emergency character of the project, the concentration of laborers was considerable, with a maximum of 1,000 people working in a very restricted area. Work was carried out by piecework contractors engaged directly by the consultant. Each of these had a labor force of approximately 50 local people. The system proved to be quite efficient in view of construction progress and cost. It is discussed further in the next section.

Although the final cost figures cannot be given at the present time, it is estimated that the overall construction cost (including design, supervision, and administrative costs) for the rehabilitation of the road stretch from Km 44.6 to Km 46.0 will amount to $5 million (U.S.). Initial construction costs of the road, updated to comparable values with average inflation rates of Nepal, were of the order of $0.55 million (U.S.) per kilometer. Without the investment of the Charnawati Rehabilitation Project, 65 km of road toward Jiri would have been degraded to a fair-weather road and the Jiri area would have been cut off for at least 5 months during the monsoon period.

DESIGN OF HILLY ROADS IN UNSTABLE AREAS

It is clear that because of the active erosion processes in the Himalayan hills and mountains, disasters like the one at Charnawati will happen again and can only be limited but not avoided by a prudent choice of road alignment. The problem is enhanced if a low-cost approach is used for the road design and construction, minimizing structures and initial investments at critical locations. Actually, the two principles of low-cost approach and minimum maintenance demand are in conflict: it has been confirmed that long-term preservation of

hilly roads can only be ensured by a substantial maintenance effort. Thus, the need to consider event-related rehabilitation methods from the beginning is imperative.

Initial Design

Extreme care must be taken when the alignment is designed for a low-cost road in terrain such as that described above. Use of risk-hazard mapping and engineering as described by Wagner et al. (*6,7*) and Fookes et al. (*8*) should be compulsory for alignment design. These techniques allow the recognition of possible trouble spots with respect to floods or slides to elaborate the proper strategy to cope with the problem and to correctly choose among the possibilities between locally high initial investment with lower maintenance or low initial investment with higher periodic and event-related maintenance.

The Lamosangu-Jiri Road was constructed with a low-cost approach, implicitly accepting the risk of damage by unusual events and leaving the task of rehabilitation to event-related maintenance. The Charnawati case showed that this event-related maintenance can be very expensive, since the rehabilitation costs for 1.4 km equaled approximately the initial construction cost of 10 km of road.

The decision on the initial approach to be taken depends not only on the funds and technology available, but also on the estimated performance of the maintenance organization. Therefore it could easily be the case that higher initial investment would be justified if the maintenance organization were not able to achieve the targets necessary to safeguard the road or if the funds available did not allow high maintenance costs. Although labor-intensive construction techniques can be very efficient and successful, it is evident that major problems of slides or river scouring are often beyond the possibilities of these techniques and call for modern methods with or without the involvement of substantial equipment.

Without giving a final recommendation (which is not possible), either the trouble spots should be treated with full commitment to the best possible solution at the initial stage or the initial investment should be kept at a low level and, most important, authorities should be made aware of the need of further event-related maintenance.

Emergency Maintenance

Emergency maintenance is defined as the maintenance operations necessary to keep the road open with acceptable serviceability after unusual events (slides, floods, accidents). As for the initial design and construction, options are open from low-cost solutions (e.g., clearing by bulldozer after slides) to higher investment as for the Charnawati Rehabilitation Project. Again, the choice of approach has a strong influence on the future maintenance demand.

Regarding the organization of the emergency work, it has been found that the approach of overall project management by a consultant with direct labor contracts is advantageous if construction time is short. A drawback of this approach is that it required the bypassing of the normal tendering procedure. Experience showed that the tendering system in Nepal is not efficient enough to cope with serious time constraints. It is thought that these considerations for the design of roads in unstable hilly areas are applicable not only to Nepal but to many other countries developing their road network under financial constraints in geologically unstable areas.

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Pavements

Existing Methods for the Structural Design of Aggregate Road Surfaces on Forest Roads

MARGOT T. Y. YAPP, JOHN STEWARD, AND WILLIAM G. WHITCOMB

The Forest Service currently uses several aggregate surfacing design methods. Current design methods range from the "best-estimate" method to techniques that were developed or adapted by the different forests or regions. These techniques have various deficiencies; the best-estimate method has been criticized by the General Accounting Office. Also, several of these methods do not provide the technical capability to analyze unusual design situations, the effects of changes in the use of Forest Service roads, or the ability to incorporate technological advances. Such problems as different levels of failure criteria, seasonal haul traffic, variable tire pressures, and others have also not been met by the USDA Forest Service *Interim Guide for Thickness Design of Flexible Pavement Structures*. To meet these needs, the Forest Service recommended that a project be initiated to develop a second surfacing guide and computer program for the structural design of aggregate-surfaced and earth roads, utilizing existing technology as much as possible. This paper focuses on a description and evaluation of nine design methods that were deemed most suitable for use in Forest Service projects. The design methods evaluated include all of the known methods currently being used within the Forest Service. The other major organization performing aggregate-surfaced or earth design, the Corps of Engineers, is also represented. Recommendations are made pertaining to the need for field studies to refine the design algorithms for aggregate-surfaced and earth roads.

Because of shortcomings in the current design methods for aggregate-surfaced roads (*1*), the U.S. Forest Service (USFS) in 1988 reviewed the current direction regarding the design of such road surfacings and produced a surfacing design evaluation report for internal use and discussion (*2*). One of the key recommendations resulting from that report was to develop a surfacing design guide for aggregate-surfaced and earth roads using existing technology.

A project was therefore initiated to develop a guide and companion computer program to assist in the structural design of aggregate-surfaced and earth roads. A Forest Service Technical Advisory Board consisting of representatives from several Forest Service regions was appointed to provide technical guidance during the project. The initial tasks of the project focused on the review of existing technology related to the structural design of aggregate-surfaced roads. The results of that review and evaluation are described and the information contained in the resulting report (*3*) is summarized. This paper is intended as a guide to the different design methods and their interrelationships. One of the objectives is to communicate to others the nature of the research in the structural design of aggregate-surfaced roads. A similar report was subsequently published by the Corps of Engineers (*4*) in 1989 after the completion of this review.

Several existing relationships were identified as potential candidates for this design guide; however, no clear choice for adoption was discovered and all design methods currently available had some serious limitations. In fact, there was considerable disappointment with the current state of existing technology. However, after considerable review, a relationship developed by the Corps of Engineers in 1978 was selected as the thickness design algorithm for inclusion in the design guide and computer program. This algorithm has been used little, if at all, in the fields; however, this is also true for most of the design algorithms reviewed. The equation was developed by researchers at the Corps' Waterways Experiment Station (WES) using previous field data. The intent during its development was to provide a relationship as a starting point that could then be refined through field experiments.

The authors would like to emphasize that the need for field studies to refine the design algorithm is still a critical item in the development of aggregate-surfacing design techniques for use by the Forest Service. Focused, small-scale field validation experiments are vital to the continued use and acceptance of any design method selected.

Nine design methods that were deemed to be most suitable for use in Forest Service projects are described. The design methods include all of the known methods currently being used within the Forest Service. The other major organization performing design of aggregate-surfaced or earth roads, the Corps of Engineers, is also represented. An evaluation of the design methods and recommendations for the selected design procedure are included. Other recommendations include the desirability of field studies for refinement of the design algorithms for aggregate-surfaced and earth roads. One such field study is the Central Tire Inflation Project at the Waterways Experiment Station in Vicksburg, Mississippi, which is a cooperative effort between the USFS, the Corps of Engineers, and FHWA.

DESCRIPTION OF DESIGN METHODS

Nine major design methods are summarized and evaluated:

1. U.S. Army Corps of Engineers Method;
2. USFS Region 4 implementation of Corps rutting equation;

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3. USFS Surfacing Design and Management System (SDMS);
4. USFS Region 8 Analysis Road Materials System (ARMS);
5. AASHTO low-volume road design method (1986);
6. USFS Chapter 50 design method;
7. USFS Region 1 seasonal surfacing method;
8. Willamette National Forest "Seasurf" design method; and
9. FHWA report.

The design methods identified above can be categorized into three logical groups. Group 1 (items 1 and 2 above) includes work performed by the U.S. Army Corps of Engineers and the USFS Region 4 implementation of one of the Corps' rutting equations. In addition, a draft version of their aggregate-surfacing design guide for roads and airfields was reviewed. The three basic reports identified in this draft report are by the Corps of Engineers (*5–7*). Remboldt of Region 4 has adapted the design equations from Technical Report S-78-8 (*5*) for use in design (Remboldt, unpublished data).

Group 2 (items 3, 4, and 5 above) may be termed the SDMS group. This includes the SDMS manual and computer methods, the Region 8 ARMS Program, and the 1986 AASHTO low-volume road design method (*8*). The ARMS program is essentially a computerization of the SDMS manual equations, while the AASHTO low-volume road design method provides nomograph solutions to the SDMS manual equations.

Group 3 (items 6, 7, and 8 above) includes three closely related methods—the USFS "Chapter 50" design method, the USFS Region 1 seasonal surfacing method, and the Willamette National Forest "Seasurf" design method. The seasonal surfacing method uses a modified Chapter 50 design to include seasonal characteristics of materials. The Willamette "Seasurf" design method is similar to the USFS Region 1 seasonal surfacing method.

The final design procedure reviewed (item 9) is a report for FHWA (*9*), which contains elements from each of the above three groups and incorporates three levels of design. Level C is the simplest design method and consists of the equation developed by Hammitt (*6*) (Group 1). Level B is a more complex level and consists of the AASHTO design procedure from the 1972 Interim Guide (*10*) for a one-layer system (Group 2). Level A is the most complex level and consists of the manual design procedure from SDMS (Group 3).

The remainder of this section provides an overall perspective of the foregoing design methods. All the design methods for aggregate-surfaced and earth roads found in the literature are generally related to each other and typically can be traced back to two basic studies: (a) the California bearing ratio (CBR) design method itself, developed by the California Division of Highways (*11*) for flexible pavements and adapted to airfield pavement design by the Corps of Engineers, and (b) the AASHO Road Test (*12*). The lineage of all subsequent aggregate-surface design methods can be traced to one or both of these studies.

Figures 1 and 2 are representative schematics of the genealogy, as it were, of these design procedures from the Corps' work and the AASHO Road Test. Figure 1 shows the roots of the CBR design method with the California Division of Highways. This method was derived from empirical studies of flexible pavement performance in the 1920s and 1930s. During World War II, the Corps of Engineers was faced with the task of designing airfield pavements for the war effort. They selected California's CBR design method as a starting point and undertook an extensive series of field studies to adapt it for airfields (*13*). The work to refine design methods for low-volume aggregate-surfaced and earth facilities has continued with the work by Hammitt (*6*). Barber et al. (*5*), and the new Corps aggregate surfacing design manual (*7*).

As shown in Figure 1, the Forest Service derived its modified *f*-factor from Hammitt's work for use in Chapter 50. This was combined with AASHTO's 1972 Interim Guide to produce the 1974 version of the Chapter 50 design method. The Forest Service Region 1 and Willamette seasonal design methods are direct descendents of the Chapter 50 design method.

Two of the three levels of design presented in the FHWA report (*9*) are also shown in Figure 1. The Level C design uses Hammitt's equation, whereas the Level B design uses the 1972 AASHTO Interim Guide design procedure for a one-layer system.

The work by Barber et al. (*5*) was a continuation of the Corps' investigations into low-volume aggregate-surfaced and earth road design. As shown in Figure 2, this research led to both the development of the manual equations in the Forest Service SDMS and the algorithms used in the computer model. Remboldt from Forest Service Region 4 also used the rutting equation to derive his design method. Both the ARMS procedure (from Region 8) and the AASHTO low-volume road design method were derived from the SDMS manual equations. Some modifications were made in ARMS, but the AASHTO guide adopted the equation with no modifications. Finally, in the report by Alkire for FHWA (*9*), these equations were used as the basis for his Level A design. Each of these design methods is described in more detail below.

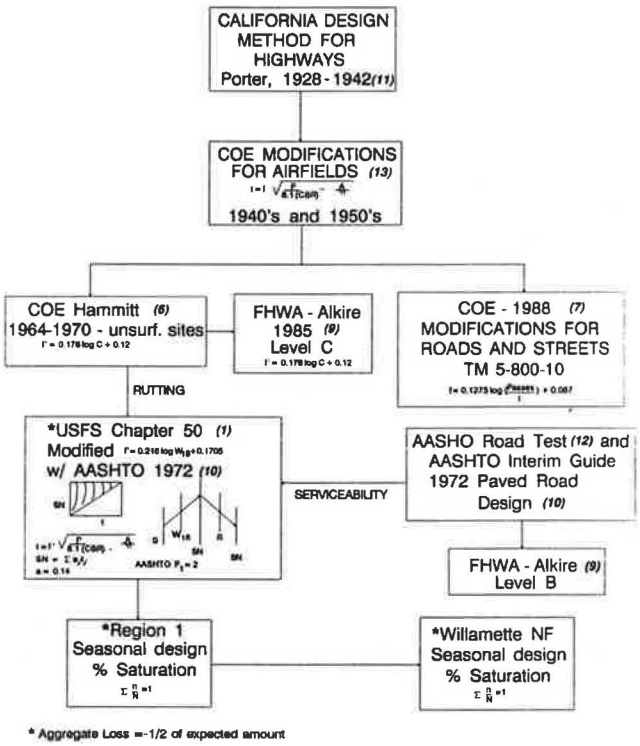

FIGURE 1 Design method relationships originating from California for aggregate-surfaced and earth roads.

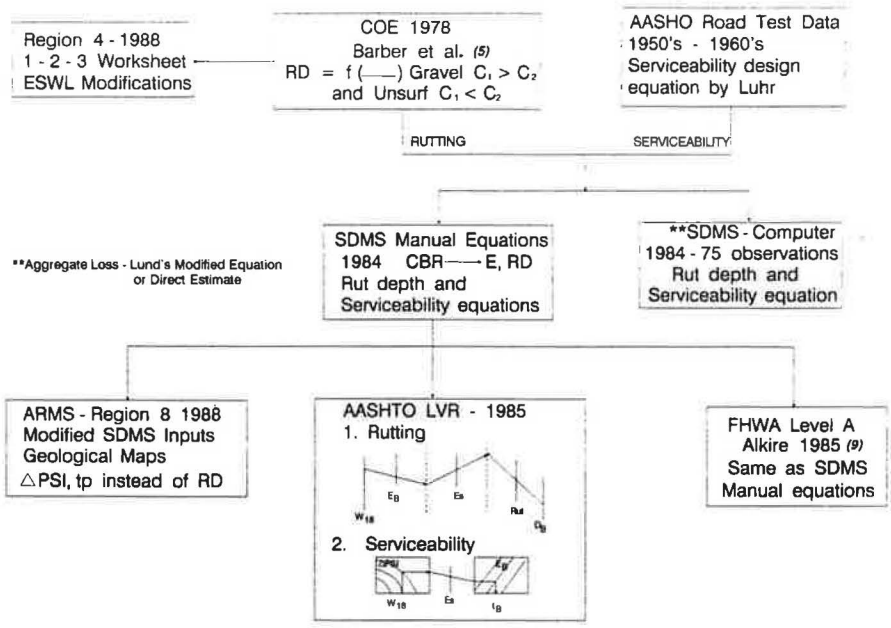

FIGURE 2 Design method relationships originating from the Corps of Engineers for aggregate-surfaced and earth roads.

U.S. Army Corps of Engineers Method

This aggregate thickness design procedure is based on the Corps' early work on flexible pavements. In the 1950s, WES developed a set of design curves for flexible pavements based on the CBR (*14*). Mathematically, the form of the equation is

$$t = f\left[\frac{P}{8.1\ (\text{CBR})} - \frac{A}{\pi}\right]^{0.5} \tag{1}$$

where

t = design thickness (in.),
f = percent of pavement design thickness,
= 0.23 log C + 0.15 (where C = coverages),
P = single or equivalent single-wheel load (lb),
A = tire contact area (in.^{2}) = (load/tire contact pressure), and
CBR = California bearing ratio of underlying material.

The design thickness (t) is the thickness of the asphalt concrete and base course that would be required to protect the subgrade having a certain strength (CBR) for a number of coverages (C) of a given load (P).

In the late 1960s, WES built test sites to develop a similar equation for determining thicknesses for unsurfaced airfields, and this is documented in Hammitt's report (*6*). The test sites contained clay materials of controlled strengths (CBR) and different depths. The failure criteria used were based on permanent deformation or rutting and elastic deformation.

The number of coverages required to cause failure was recorded. Equation 1 can be solved for f since t, P, CBR, and A are all known. Because the test sites were unsurfaced, f is referred to as f', or the ratio of the unsurfaced thickness to the flexible pavement thickness:

$$f' = t\left[\frac{P}{8.1\ (\text{CBR})} - \frac{A}{\pi}\right]^{-0.5} \tag{2}$$

It was then possible to plot f' against the failure coverage and, through linear regression analysis, obtain the following relationship:

$$f' = 0.176 \log C + 0.12 \tag{3}$$

where the terms are as previously defined. Substituting f' for f in the original thickness equation (1) results in the following:

$$t = (0.176 \log C + 0.12)\left[\frac{P}{8.1\ (\text{CBR})} - \frac{A}{\pi}\right]^{0.5} \tag{4}$$

This expression then determines the thickness of the cover material for unsurfaced roads required to prevent subgrade failure. The strength of the cover material is then determined (*14*). The tire pressure, wheel load, and number of coverages are required to determine the CBR of the cover material.

In 1978 another study was published at WES by Barber et al. (*5*), which led to additional design equations for aggregate-surfaced roads. Existing rutting data from previous work at WES for earth, gravel-surfaced, and flexible pavements were utilized to develop deterioration and reliability models. Once the models had been developed, computer programs were written. The models all predicted rut depth of the surface layer given the load, tire pressure, surface layer thickness,

and strengths of the material layers in CBR. This study also recommended field studies to validate the relationships developed; however, conversations with Barber in 1988 indicated that the field studies were not performed.

The model developed for aggregate-surfaced roads is shown below and assumes that the top layer is stronger than the bottom layer ($C_1 > C_2$):

$$RD = 0.1741\left[\frac{P_k^{0.4704} t_p^{0.5695} R^{0.2476}}{(\log t)^{2.002} C_1^{0.9335} C_2^{0.2848}}\right] \quad (5)$$

where

RD = rut depth (in.),
P_k = equivalent single-wheel load (ESWL) (kips),
t_p = tire pressure (psi),
t = thickness of top layer (in.),
R = repetitions of load or passes,
C_1 = CBR of top layer, and
C_2 = CBR of bottom layer.

In 1988, the Corps aggregate-surfacing design procedure (7) was brought in line with the existing Corps flexible pavement procedure in the interests of consistency. Figure 3 is from the 1988 draft version of Technical Manual 5-822-30 (7). The design equation is

$$t = \left[0.1275 \log\left(\frac{\text{Passes}}{I}\right) + 0.897\right] \times \left[\frac{\text{ESWL}}{8.1\,(\text{CBR})} - \frac{A}{\pi}\right]^{0.5} \quad (6)$$

where

Passes = no. of repetitions of 18-kip single-axle loads;
I = traffic index, a value of 2.64 is used for 18-kip dual wheel single-axle loads;
ESWL = equivalent single wheel load (lb);
CBR = California bearing ratio of underlying material; and
A = contact area of one tire (in.^{2}).

This equation is similar to Equation 1, with the exception that the f-factor has been modified. In conversations with Donald Ladd at WES in 1989, it was determined that the f-factor was developed from test data that had not been published. Tests had been performed on aggregate-surfaced roads, but funds for that project were subsequently cancelled, hence the lack

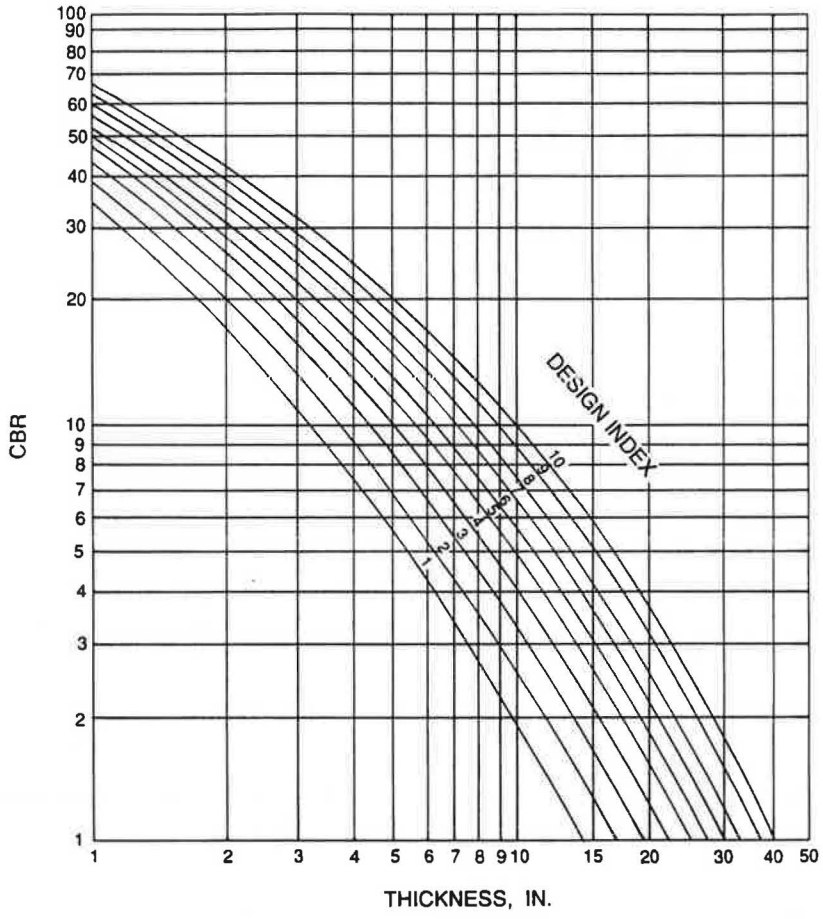

FIGURE 3 Design curves for gravel-surfaced roads (7).

of published information. At this time, the failure criteria associated with this design equation are unknown.

USFS Region 4 and Corps Rutting Equation

Rutting was selected as the failure criterion for aggregate road design in Region 4 (Remboldt, unpublished data). The design equation used in this procedure comes from work by Barber et al. (*5*) described previously (Equation 5). To obtain the ESWL, Region 4 has derived a regression model for use as follows:

$$\text{ESWL} = C1 * \text{Lg} + C2 * \text{Lg} * \text{Da} \quad (7)$$

where

$C1$ = 0.3209 (single axles) or 0.1646 (tandem axles),
$C2$ = 0.0151 (single axles) or 0.0127 (tandem axles),
Lg = group load (kips), and
Da = depth of aggregate (in.).

This equation is not based on the AASHO Road Test data nor that of Chapter 50. It is a new relationship based on regression equations that relate ESWL with actual load configurations. Remboldt indicates that the calculations used elastic-layered equations developed by Ahlvin and Ulery (*15*).

USFS Surfacing Design and Management System

The SDMS project evolved over a period of 15 years (1972–1987) and was known by several names, including Pavement Design and Management System (PDMS), Low Volume Roads (LVR), and SDMS. In 1972 the Forest Service and the University of Texas initiated a cooperative study to develop a pavement management system for the Forest Service road

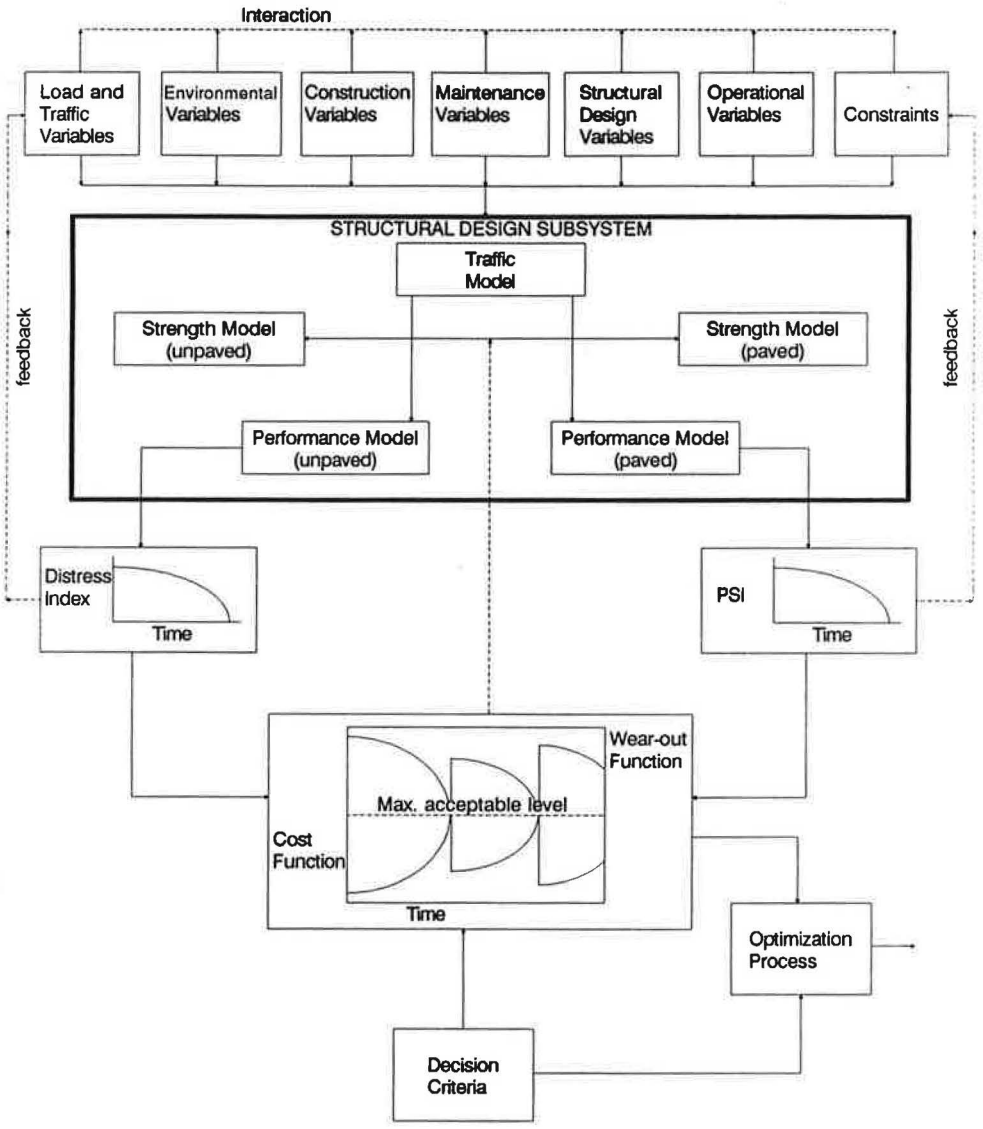

FIGURE 4 Conceptual system for low-cost roads (*16*).

TABLE 1 CLASSIFICATION OF INPUT VARIABLES FOR LVR

CATEGORY OF INPUT VARIABLES	SPECIFIC INPUT VARIABLES
MISCELLANEOUS INPUT	Problem Identification Output Formats Total Number of Materials Available Length of Analysis Period Width of Each Lane Number of Cards with Time Dependent Variables Interest Rate Road Type (Asphalt Concrete, Aggregate, Bituminous Surface Treatment) Number of Performance Periods Flag for Calculation of User Delay Cost
PERFORMANCE VARIABLES	Regional Factor Initial Serviceability Index Terminal Serviceability Index Serviceability Following Rehabilitation Non-Traffic Terminal Serviceability Rate of Non-Traffic Serviceability Reduction Variables for Aggregate Loss Equation
TIME DEPENDENT VARIABLES	Traffic (Load and Frequency) Maintenance Costs Aggregate Loss
PERFORMANCE PERIOD VARIABLES	Length of Performance Periods where Performance Period is Defined as Time Between: 1) Initial Construction and First Rehabilitation. 2) Two Major Rehabilitations. 3) Initial Construction and Subsequent Construction Changing the Surface Type.
CONSTRAINT VARIABLES	Maximum Construction Cost Maximum Total Thickness Minimum Thickness of Individual Rehabilitation Maximum Thickness of All Rehabilitations Maximum Thickness of Individual Rehabilitation Minimum Thickness of Top Layer Aggregate Loss from Erosion (Added to Other Aggregate Loss)
REHABILITATION PARAMETERS FOR CALCULATING USER DELAY COSTS	Distance Over which Traffic is Slowed Percent of ADT Through Rehab Zone each Hour Percent Vehicles Stopped Average Delay Average Speed Approaching and Passing Through Rehab Area Model to Use in Calculating User Delay Costs
GRADING AND SEAL COAT VARIABLES	Grader or Seal Coat Passes Speed of Grader and Trucks Distance Cars Follow Grader Cost of Grading or Seal Coat Time Between Gradings and Seal Coats
VEHICLE OPERATION COSTS	Average Operating Costs ($/mi) for Log Trucks and Other than Log Trucks
MATERIAL VARIABLES	Material Description Cost (in place) per cy. Layer Coefficient Minimum and Maximum Layer Thickness Salvage Value Soil Support (for Subgrade)

network. The work was conducted in three phases and three reports were produced:

- Phase I: conceptual study (*16*),
- Phase II: working model (*17*), and
- Phase III: implementation (*18*).

PDMS

In Phase I, the feasibility of developing such a pavement management system was analyzed. This led to the development of a working computer-based model during Phase II. The working model was known as PDMS. In Phase III, the experiences derived from a trial implementation in several offices of the Forest Service were presented.

As a result of this third phase, two additional projects were conducted at the University of Texas. The first of these dealt with the revision of the actual design procedure used in the Forest Service and the integration of PDMS in the U.S. Forest Service road design handbook (*18,19*). The second project was also a three-phase study to develop a data base pertaining to the design and performance of aggregate-surfaced roads. Phase I of this second project analyzed the feasibility of a

data base for PDMS (*20*), and Phase II was a pilot study (*21*) designed to field test alternative types of equipment for measuring the variables involved, recommend optimum test section length and measurement frequencies and hardware and software requirements for computerization of the data base, and develop cost estimates for several scenarios to develop the data base. In Phase III, field data were collected during 1984 and 1985 and regression analyses performed (*22*).

LVR

The development and implementation phases of LVR are documented elsewhere (*17,18*). The intent of the LVR program was to computerize the existing Forest Service design procedures (Chapter 50) (*1*) within the conceptual model presented in the Phase I report (*16*). This conceptual model is shown in Figure 4.

The structural design subsystem was the existing Chapter 50 design. The LVR program was written to computerize this design method. The program would generate pavement structures that would meet the performance limitations and constraints provided. As design structures that met these criteria were identified, they were saved and listed in order of increasing total cost. The input variables for LVR are shown in Table 1.

SDMS

The work to develop SDMS took place over a 5-year period (1978–1983) and has been documented best by Luhr (*23*). By 1978 LVR was operating successfully. However, in July 1978, the Forest Service and the University of Texas entered into a cooperative agreement to revise and improve Chapter 50 of the Forest Service Transportation Engineering Handbook. The objective was to improve the flexible pavement design guide by making needed revisions and provide additional capabilities by accomplishing the tasks shown in the text box.

These modifications initially resulted in PDMS. However, during the performance of these tasks, it became apparent that Chapter 50 was inadequate to meet the objectives shown in the text box. Therefore, a new structural design algorithm was required. Subsequently, Luhr (*23*) reviewed the literature and concluded that no existing design procedure was available that addressed the desired capabilities. A decision was then made to develop a new procedure using a combination of theoretical and empirical methods to specifically address the needs of the Forest Service. This new design method became known as SDMS.

In the initial development of the SDMS equations, Luhr (*23*) used an elastic-layered program known as ELSYM5 to analyze the AASHTO Road Test information and concluded

Tasks to be accomplished during SDMS project (1978-1982).

(1) Develop reliability factors for different classifications of roads. Provide the designer with flexibility in accordance with road's present and future classifications. Provide the means for a designer to select appropriate failure criteria for each specific project.

(2) Improve the Rutting Model using a more rational approach. Improve the Aggregate Loss Model in accordance with recent research experience.

(3) Analyze the wheel load equivalencies for single and dual wheels for legally loaded and oversized loads as they presently appear in Chapter 50. Analyze recent information on structural layer equivalencies. Revise the wheel load equivalencies and the structural layer equivalencies in Chapter 50 in accordance with these analyses.

(4) Develop a procedure in Chapter 50 for determining the regional factor.

(5) Provide improved procedures for evaluating the amount, type, frequency, and distribution of traffic over the design life of the roadway surface.

(6) Provide a capability to consider the effects or non-effects of roadway drainage.

(7) Provide a deflection design alternate in Chapter 50.

(8) Analyze the findings from recent pavement and other appropriate research for low-volume roads. Incorporate those significant findings that area compatible with the basic approaches followed in Chapter 50. Such items as vehicle operating cost models and selected findings from the Brazil study associated with the University of Texas should be carefully scrutinized to determine their applicability and potential for enhancement of Chapter 50.

(9) Provide a comprehensive, interactive connection between Chapter 50 and the LVR computer program. Incorporate in Chapter 50 an up-to-date LVR users manual with all appropriate discussion, instruction, figures, tables, and examples of input and output data. In addition, make the appropriate connections and referrals in the LVR users manual with the revised Chapter 50.

(10) Check all figures, tables, nomograph charts, and other supporting material in Chapter 50 for correctness, accuracy, and ease of use. Make changes as needed. Completely rewrite Chapter 50, with an appropriate explanatory text to reflect all of the aforementioned changes, additions, and improvements.

that vertical compressive strain at the top of the subgrade was found to be the most promising parameter to correlate with the Pavement Serviceability Index (PSI). The following equation (eventually used as one of the "manual" equations for SDMS) was obtained from linear regression techniques:

$$\log W_{tx} = 2.15122 - 597.622(\varepsilon_{SG}) - 1.32967 (\log \varepsilon_{SG}) + \log\left(\frac{4.2 - P_t}{4.2 - 1.5}\right) \quad (8)$$

where

W_{tx} = number of applications of axle load x,
ε_{SG} = vertical compressive strain at top of subgrade, and
P_t = terminal value of PSI (terminal serviceability).

For aggregate-surfaced roads, performance models for rutting and aggregate loss were also introduced. The rutting model was obtained using Corps of Engineers data (*5*). Luhr indicates that the equation for the manual design was developed using all the data points from the Corps 1978 report. He also indicates a certain lack of confidence in the published Corps regression equations. The rutting equation is

$$W_{18R} = 0.1044 * \text{RUT}^{2.575} * \log_{10} \text{THICK}^{5.155} * \left(\frac{E_1}{1{,}800}\right)^{3.434} * \left(\frac{E_2}{1{,}800}\right)^{1.048} \quad (9)$$

where

W_{18R} = no. of applications of 18-kip equivalent single-axle loads (ESALS),
RUT = rut depth (in.),
THICK = thickness of aggregate surface (in.),
E_1 = modulus of aggregate surface (psi), and
E_2 = modulus of roadbed (psi).

In 1979, during the Low-Volume Road Conference in Ames, Iowa, a decision was made to incorporate this algorithm as a manual procedure in the "new" Chapter 50 (*17*). This and the development work to computerize the entire design process were completed in 1983. During the ensuing 5 years, significant changes in computer technology and existing design methods (1986 AASHTO guide) prompted the Forest Service to reevaluate the situation. In 1988 the Surfacing Design Evaluation Report (*2*) was written for internal use and discussion and recommended the following:

1. Adopt the revised 1986 AASHTO design guide and the companion program (DNPS86/PC) for bituminous-surfaced roads,
2. Develop a surfacing design guide for aggregate-surfaced and unsurfaced roads using existing technology, and
3. Incorporate the concept of multiple user levels into the design process (the user levels imply differing levels of complexity of operation and variability of design).

USFS Region 8 ARMS Method

ARMS is a road surfacing design method developed in Region 8 of the Forest Service (Scholen, unpublished data). It was a result of the need for an aggregate management system suitable for a regionwide situation in which there were many isolated, low-cost, short road segments. The ARMS surfacing design procedure utilizes existing geologic data (such as state geological maps) as an index to soil properties for input to surface design equations. Because of oil and gas exploration over much of Region 8, detailed geologic maps are available. The maps separate rock formations into units of similar petrographic characteristics, and these characteristics are then correlated to engineering properties of soils. In this way, the maps are used to establish a geotechnical data base for use in ARMS. In addition, regression equations developed from correlated laboratory data are used to supplement traditional sampling and testing techniques.

The thickness design is based on the SDMS manual equations. Two criteria are used—rut depth and serviceability loss. The design equation based on rut depth is

$$W_{18RUT} = 64.51 * (t_p)^{-1.4665} * [3 (\text{TSI})^{-0.5}]^{2.575} * \log t^{5.155} * \left(\frac{E_B}{1{,}800}\right)^{3.434} * \left(\frac{E_{SG}}{1{,}800}\right)^{1.048} \quad (10)$$

where

W_{18RUT} = no. of applications of 18-kip ESALs,
t_p = tire pressure (psi),
TSI = terminal serviceability index,
t = thickness of aggregate surface (in.),
E_B = resilient modulus of aggregate surface (psi), and
E_{SG} = resilient modulus of subgrade (psi).

Equation 10 is very similar to the SDMS manual rutting equation (Equation 9). It can be seen that the last three terms are the same. The rut depth term in Equation 9 was replaced with an expression that uses terminal serviceability index instead. Region 8 has developed Table 2, which relates the rut depth as a function of Traffic Service Level (TSL). The appropriate TSI and its corresponding allowable rut depth are selected from Table 2 and entered into Equation 10.

The other criterion used in this design procedure is serviceability loss, and this is identical to the SDMS Equation

TABLE 2 SERVICEABILITY INDEX AND RUT DEPTH

TSL	PSI	TSI	MRD
A	4.7	2.0	2.1
B	4.2	1.5	2.4
C	3.5	0.5	4.2
D	3.5	0.25	6.0

Notes:

TSL = Traffic Surface Levels
PSI = Present Serviceability Index
TSI = Terminal Serviceability Index
MRD = Maximum allowable rut depth

8. The ARMS program allows the user to enter resilient modulus data for both aggregate and subgrade layers, if available. If these are not known, a set of default values is available for different soil types such as crushed limestone, limerock, and sand clay. Initially, moduli were calculated on the basis of CBR using the following equation, which is found in SDMS:

$$M_R = 1{,}800\text{CBR}^{0.7} \tag{11}$$

where M is the resilient modulus in pounds per square inch and CBR is the California bearing ratio.

In 1986 laboratory testing was performed on typical aggregates used in Region 8 to verify the assumed values. In most cases, the values were verified, but some were modified to reflect the laboratory results. Variations in rainfall are also considered through the number of rain days a month. Traffic is assumed to be uniform throughout the month, but the program will consider rainfall effects (and therefore weaker subgrade) in its design based on the amount of traffic hauled during rainy days. The default tire pressure is 80 psi, but other values may be entered, with the recognition that the higher tire pressures drastically increase the rate of surface failure.

AASHTO Low-Volume Road Design Method

The AASHTO (1986) design guide presents graphical solutions based on the SDMS manual equations (Equations 8 and 9). No modifications were made to these equations apart from converting them to nomographs. Figures 5 and 6 show the two nomographs used for design.

However, aggregate loss has been considered. If aggregate loss is significant, an estimate may be made and added to the aggregate thickness:

$$D_{BS} = \overline{D}_{BS} + (0.5 \times \text{GL}) \tag{12}$$

where

D_{BS} = total thickness of aggregate layers (in.),
$\overline{D}_{BS}$ = thickness of aggregate layer based on rut depth or serviceability loss (in.), and
GL = estimated gravel loss over performance period (in.).

USFS Chapter 50 Design Method

The current version of Chapter 50 was developed in 1974 and last revised in May 1982 by the office in Region 6 (*1*). The 1982 version was intended as an interim measure until SDMS was completed. Two sources were used to develop the design procedure. The first is the Corps of Engineers, specifically the work done by Hammitt (*6*) on unsurfaced roads described previously. Hammitt's work involved the construction of unsurfaced test sites and the development of a modified f-factor through regression analysis. The failure criterion used was a 3-in. rut depth. On the basis of the same data, the Forest Service then similarly developed a modified f, but used a 2-in. rut depth as the failure criterion instead. The modified f that was obtained is

$$f = 0.216 \log C + 0.1705 \tag{13}$$

where C is coverages.

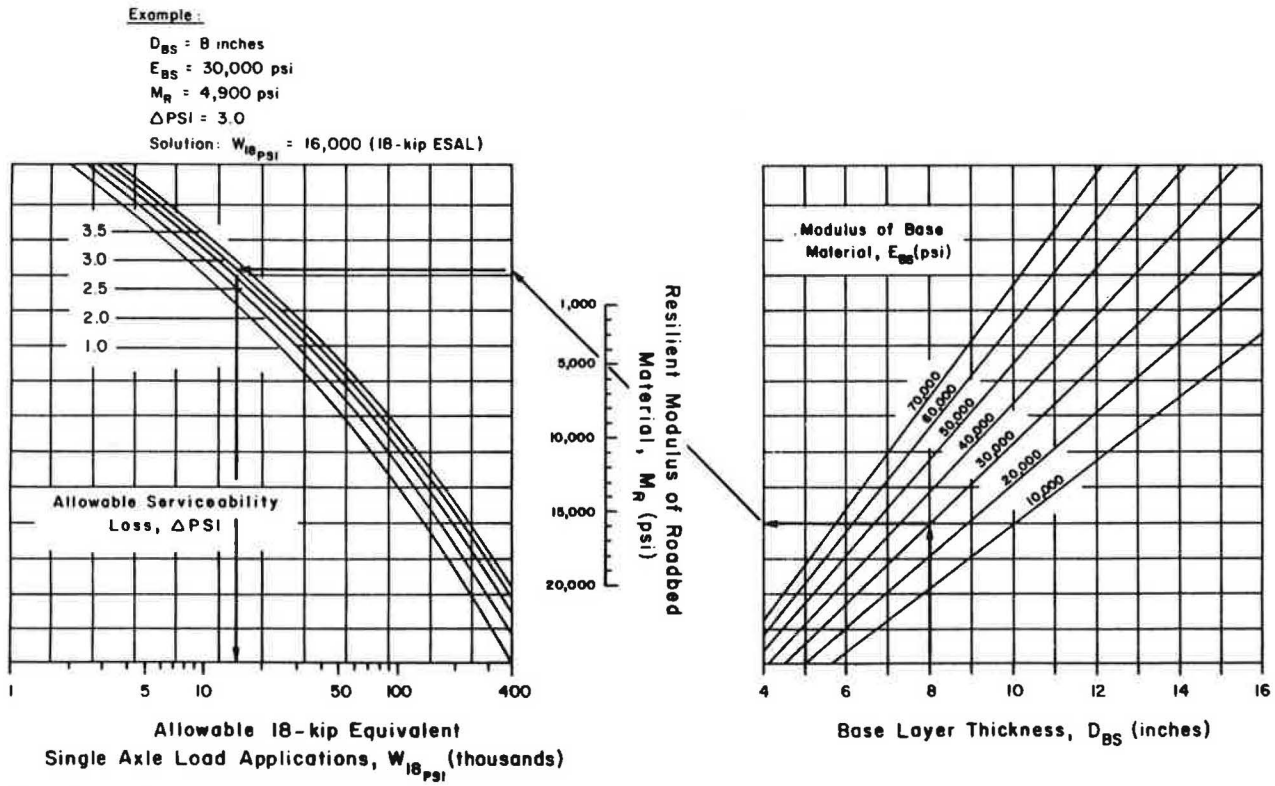

FIGURE 5 Design chart for aggregate-surfaced roads considering allowable serviceability loss (*8*).

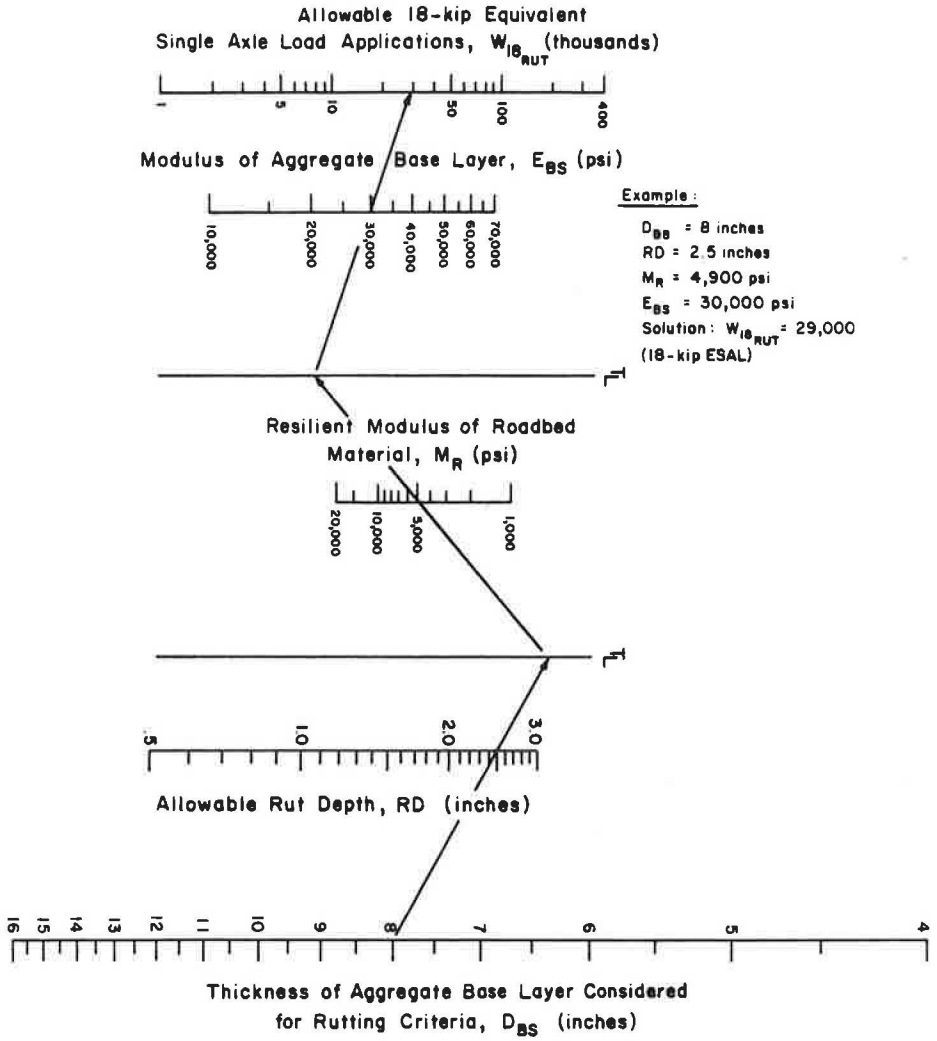

FIGURE 6 Design chart for aggregate-surfaced roads considering allowable rutting (*8*).

The other source for the design method came from the AASHTO Interim Guide (*10*), originally published in 1972 and subsequently revised in 1981 (the Blue Book). The procedure for determining the weighted structural number was incorporated into Chapter 50 in 1974 with minor modifications to the nomograph. Specifically, the modifications were aimed at extending the scale for the regional factor (R) to include values greater than 5. The design equation for this nomograph is as follows:

$$\log W_{t18} = 9.36 \log(\text{SN} + 1) - 0.20 + \frac{G_t}{0.40 + \frac{1{,}094}{(\text{SN} + 1)^{5.19}}} + \log \frac{1}{R} + 0.372(S - 3.0) \quad (14)$$

where

W_{t18} = total applications of 18-kip ESALs,
SN = structural number,
S = soil support value,
R = regional factor, and
G_t = $\log[(4.2 - P_t)/(4.2 - 1.5)]$ where P_t is the terminal serviceability index.

Figure 7 was then developed for a given aggregate layer coefficient (a) of 0.14. The curved lines represent the Corps equations (1, 2, and 3), and the straight diagonal line is that from AASHTO (Equation 14). The curve giving the lesser aggregate thickness controls. If material with a lower layer coefficient is used, the depth must be adjusted using the following equation:

$$\text{SN} = a_1 D_1 + a_2 D_2 + \ldots \quad (15)$$

where

SN = structural number,
a_1 = layer coefficient for the ith layer, and
D = thickness of ith layer

This last step is an unusual feature of this method, particularly if the controlling thickness was originally determined using the Corps equation.

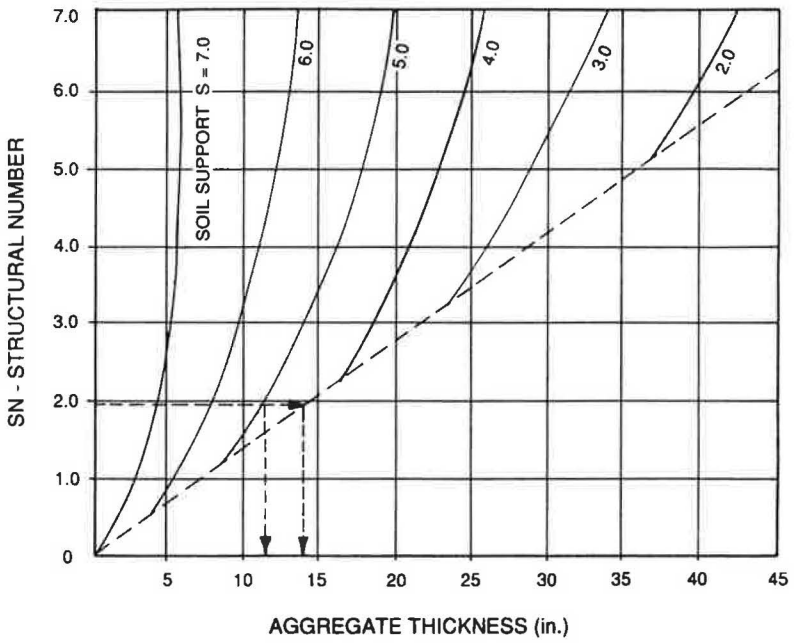

FIGURE 7 Design chart for aggregate-surfaced pavements (*1*).

USFS Region 1 Seasonal Surfacing Method

In 1980 Region 1 of the Forest Service developed a design procedure for aggregate roads based on seasonal haul (*24*). This was referred to as the Region 1 Supplement No. 10 to the Forest Service Handbook (FSH 7709.11), Chapter 50. The modifications considered seasonal variations in subgrade soil strengths in the design of aggregate-surfaced roads. The modifications could be used for shutting down haul during spring thaw, restricting haul to dry or frozen subgrades, or compacting subgrades to exceed standard specifications. In addition, a terminal serviceability index of 0 instead of 2 was used in the design charts.

Two studies were used to monitor subgrade soil conditions and the performance of aggregate-surfaced roads. The results were used to obtain a relationship between percent saturation of the subgrade and time of the year. First, a relationship was established between subgrade soil strength, percent saturation, and percent compaction (by laboratory testing). Next, the seasonal variation in percent saturation throughout the design period (usually one year) was estimated from field data. Finally, the design period was divided into time periods of similar percent saturation and a CBR assigned to each increment.

Once the above steps have been completed, the aggregate thickness may be estimated for the anticipated hauling season. These roads designed with this method are for use only during certain times of the year, and are not all-weather roads. They will appear to fail prematurely if used for circumstances other than those considered in the design. Also, the procedure was developed primarily to restrict haul to periods when the subgrade is strongest; therefore, the converse (i.e., design for when subgrades are weakest, such as during spring thaw) may not necessarily be a valid design approach.

Willamette National Forest "Seasurf" Design Method

The Willamette design method (unpublished data, 1988) was also modified from the original Chapter 50 (*1*). The modifications incorporated seasonal changes in subgrade soil strengths and traffic. In addition, Miner's hypothesis is used to sum up damage ratios in the design of aggregate thickness. The procedure is very similar to that described for Region 1, if not the same. The regional factor used, however, is different for the Willamette. In addition, Region 1 uses a terminal serviceability index of 0, while the Willamette uses 2 or 1.

FHWA Report

Alkire (*9*) recently published a report sponsored by FHWA for the design and operation of aggregate-surfaced roads. For aggregate thickness design, Alkire provides three levels of design complexity—A, B, and C—with C being the least complex and A the most complex. Each level utilizes design methods that have generally been discussed previously. Therefore, only the key elements of the design will be highlighted.

Level C Design

The Level C design uses the rut depth model developed by Hammitt (*6*) and is the simplest design procedure. Alkire assumes that the coverages are equal to passes and that the equivalent single-wheel load is 9,000 lb for an 18-kip axle load. Implied in the use of this equation is the acceptance of the 3-in. rutting failure criterion. The procedure was further

simplified when he suggested that the effect of traffic could be removed and thickness could be calculated using

$$t = (750/\text{CBR})^{0.5} \quad (16)$$

Level B Design

The Level B design is the AASHTO procedure taken from the 1972 Interim Guide (*10*). Figure 8 shows the relationships developed between 18-kip ESALs and structural number for various soil support values. To develop this chart the following assumptions were made: initial serviceability = 4.2; terminal serviceability = 1.5; and regional factor = 1. By using this chart, traffic may be calculated for a given thickness or thickness can be determined from estimated traffic. The relationship between structural number and thickness was shown previously in Equation 15. Tabular values are also provided to adjust the layer coefficient on the basis of the strength of the granular material. In addition, guidance is given for the calculation of soil support and *R*-values.

Level A Design

The Level A design consists of the manual equations (Equations 8 and 9) from SDMS. Although aggregate loss is mentioned as a factor that needs to be incorporated into the design, no guidance is given regarding methods for estimating this factor.

Finally, Table 3 is a summary of all the design methods reviewed and their respective equations.

Aggregate Loss

Aggregate loss and its prediction are important factors in aggregate surfacing design and maintenance. One equation that was used in the 1977 version of LVR was developed by Lund (*25*) and is as follows:

$$\text{GL} = 0.162 + 0.0188(\text{LT}) + 0.0382(\text{F/C}) - 0.00110(\text{TTU}) - 0.00213(\text{P3/4}) \quad (17)$$

where

GL = aggregate loss corrected for settlement (ft),
LT = number of load logging trucks (000s),
F/C = fill or cut section (fill = 1.0, side cast slope = 1.5, cut = 2.0),
TTU = total two-way traffic units (000s) (one non-truck = 2TTU, one logging truck round trip = 10 TTU), and
P3/4 = percent of road surfacing sample smaller than 0.75 in. in diameter.

Later this equation was simplified in the 1979 LVR version to

$$\text{GL} = 0.1 + 0.01019(\text{LT}) \quad (18)$$

where terms are as identified above (*25*,*26*). A good discussion of the Lund equations is contained in the report by McCullough and Luhr (*18*).

Equation 18 was used in SDMS. In addition, the SDMS user's manual provided other equations that could be used to calculate the aggregate loss manually. The first equation was developed in Brazil and the second in Kenya. The Brazilian equation is

$$\text{GLIN} = \frac{B}{25.4}0.0045(\text{LADT}) + \frac{3{,}381}{R} + 0.467(G) \quad (19)$$

where

GLIN = aggregate loss during time period considered (in.),
B = number of bladings during time period considered,

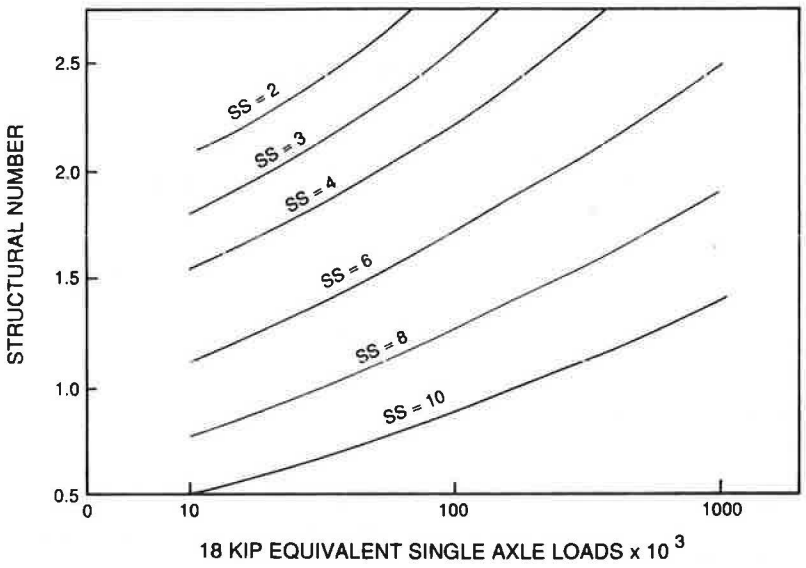

FIGURE 8 Structural number for various 18-kip equivalent axle loads and soil support values (*9*).

TABLE 3 SUMMARY OF DESIGN EQUATIONS

Methods	Design Equation	Design Criteria
Corps of Engineers		
Hammitt (1970) *(6)*	Equation 4	Rut Depth
Barber et al. (1978) *(5)*	Equation 5	Rut Depth
Draft TM 5-822-30 (1988) *(7)*	Equation 6	Unknown
USFS		
Region 4	Equation 7	Rut Depth
SDMS (Manual Equations)	Equation 8	SI
	Equation 9	Rut Depth
Region 8 (ARMS)	Equation 10	Rut Depth & SI
Chapter 50	Equation 13	Rut Depth
	Equation 14	SI
Region 1	Chapter 50 w/modifications	SI
Willamette NF	Same as Region 1	SI
FHWA		
Level A	Same as SDMS Manual Equations	
Level B	Figure 8	SI
Level C	Equation 16	Rut Depth
AASHTO	Same as SDMS Manual Equations	

SI = Serviceability Index

LADT = average daily traffic in design lane (for one-lane road use total traffic in both directions),
R = average radius of curves (ft), and
G = absolute value of grade (%).

The Kenyan equation (*27*) may have more applicability in situations where logging activity is relatively minor:

$$\text{AGL} = f\left(\frac{T^2}{T^2 + 50}\right) * 4.2 + 0.092(T) + 0.0889(R^2) + 1.88(\text{VC}) \quad (20)$$

where

AGL = annual aggregate loss (in.),
T = annual traffic volume in both directions of vehicles (000s),
R = annual rainfall (in.),
VC = average percentage gradient of the road, and
f = 0.037 for lateritic gravels, 0.043 for quartzitic gravels, 0.028 for volcanic gravels, and 0.059 for coral gravels.

Recent research in South Africa (*28*) has provided another relationship for aggregate loss:

$$\text{GL} = D[\text{ADT}(0.047 + 0.0027 * N - 0.0005 * \text{P26}) - 0.365 * N - 0.0014 * \text{PF} + 0.048 * \text{P26}] \quad (21)$$

where

GL = gravel loss;
D = time period under consideration (days/100);
ADT = average daily traffic;
N = Weinert N-value (related to three climatic levels; precipitation and evaporation are related to the geotechnical behavior of the materials);
PF = plastic limit × percent passing 200-mesh sieve (P75); and
P26 = percent passing the 26.5-mm sieve.

Finally, Paterson (*29*) has reviewed a number of gravel loss relationships. Table 4 shows a comparison of gravel loss rates from various studies for a range of gradient and rainfall. As can be seen, using data from other studies for the Forest Service situation can be risky, particularly for Region 6. The combinations of gradient and rainfall do not begin to resemble the road network in Region 6. Gradients of 6 to 8 percent are common, together with annual rainfalls of 1500 to 2500 mm. Paterson has included a general model to include the major traffic, gradient, and rainfall effects, but material properties are excluded:

$$\text{GL} = (30 + 180 * \text{MMP} + 72 * \text{MMP} * G) * h * \text{ADT} * \Delta t * 10^{-5} \quad (22)$$

where

GL = surface material loss (mm),
MMP = mean monthly precipitation (m),
G = average longitudinal gradient (%),
h = proportion of heavy vehicles in traffic (fraction),
ADT = average daily traffic (vehicles/day),
Δt = time period (days).

TABLE 4 COMPARISON OF GRAVEL LOSS RATES FROM VARIOUS STUDIES (*29,30–34*)

Study Location	Units	Rate of gravel loss (mm per 100,000 vehicle units) for gradient (%) and annual rainfall (mm/yr)				Percentage heavy vehicles
		0%		3%		
		1,000	2,000	1,000	2,000	
Brazil	V	30	32	39	43	50
Ghana	HV	(20-100)	(10-60)	40-160	(30-60)	-
	V	30	13	41	26	50
Oregon	AV	--	--	--	(240)	100[1/]
	V	--	--	--	(40)	50
Kenya A	V	(7)	(21)	(12)	(60)	(35)
	V	10	30	17	86	50
Kenya B	V	10	29	16	82	-

1/ Assumed value.

Notes: V = per 100,000 vehicles; HV = per 100,000 heavy vehicles when light vehicles are present but not counted; AV = articulated logging vehicles, rated as AV = 3, HV = 6, V. -. Not available. ( ). Original data not adjusted for vehicle mix.

The general effects of material properties are understood to behave as follows:

1. Increasing the plasticity index reduces the loss rate (Brazil, Ghana),
2. Increasing the relative compaction of the surfacing reduces the loss rate significantly (*25*), and
3. Coral gravels have 40 percent higher loss rates than lateritic, quartzitic, and sandstone gravels, which in turn have 40 percent higher loss rates than volcanic (vermicular) gravels (Kenya).

EVALUATION OF DESIGN METHODS

The Forest Service evaluation report of June 1988 (*2*) provided the majority of the evaluation criteria used in this paper. These criteria may be expressed as a series of questions:

- Is the design procedure valid for aggregate-surfaced and earth roads?
- Are the inputs expected to have a major role in pavement deterioration?
- Are standard traffic units (e.g., 18-kip ESALs) used?
- Can tire pressures be varied?
- Is the material characterization "reasonable"?
- Are risk and reliability concepts considered?
- Can failure criteria levels be changed?
- Is seasonal haul incorporated into the design?
- Has there been any field experience?

The one criterion not included in the Forest Service report list that is included above is the validity of the design method. These items are discussed further in the following paragraphs.

Validity

The validity of the design method for either aggregate-surfaced or earth roads is probably the most important attribute to be considered. By validity is meant whether the basis of the development of these design procedures is appropriate for their purpose. As an example, it could be considered inappropriate to design an aggregate-surfaced road using algorithms developed from earth road data or asphalt-surfaced road data.

Input Variables

The type and quantity of input variables are critical to the success of any design procedure. Inputs that are reasonable and recognizable to the road designer, such as CBR to describe soil strength, are highly desirable. They should be in standard units of measurement and should also be readily available. Inputs such as the internal angle of friction and approach speed of a vehicle behind a grader are either difficult to obtain, esoteric, or may seem completely unrelated to the problem at hand. The consequence of such inputs is a decrease in the usefulness of the design method to the engineer or road manager.

Traffic

Traffic data used should be in standard units, such as 18-kip ESALs, or in a format that may be easily modified to ESALs, such as timber volume (board-feet). Some design methods use units that are not easily converted into ESALs, such as the Corps of Engineers Design Index (DI). The number of

repetitions versus coverages should also be noted, because they may not necessarily mean the same thing.

Tire Pressures

In recognition of the fact that a wide range of tire pressures (as wide as 50 to 150 psi) may be used in trucks, design procedures should be able to incorporate this feature. This is particularly important because high tire pressures have a dramatic effect in the deterioration of pavements. In addition, the Forest Service is currently conducting the Central Tire Inflation (CTI) study, which is evaluating the effects of low tire pressure on forest roads. It would be useful if the results of this study could be incorporated into the selected design procedure.

Materials Characterization

It is desirable that the materials used in the road structure be characterized in units of measurement that are standard and easily quantifiable. An example would be the use of the soil support value (S) or resilient modulus for soil strength, because these are widely known and may be available. Atterberg limits, although also a standard unit of measurement in soil mechanics, cannot be considered a standard parameter for the determination of soil strength in the design of roads.

Risk and Reliability

Risk and reliability may be defined in many ways. However, to evaluate the different procedures, the question posed was, "Are risk and reliability explicitly incorporated into the design method?" Currently, only the SDMS computer algorithms and the Barber et al. (*5*) models contain this variable. Reliability, as defined by AASHTO (*35*), is the probability that designed pavement sections will withstand the actual number of ESALs that will be applied over the design period. The method used by AASHTO is to apply a reliability design factor, F_R, to the design traffic and in this way modify the thickness requirements.

Failure Criteria

The failure criteria used in most of these design methods are either rut depth or serviceability loss. An additional criterion used in several other design methods is aggregate loss. Some design procedures have developed design equations that assume a given failure level such as a 2-in. rut depth, and these may not be modified. In other methods, the failure levels are variables and may be changed very easily. It would be desirable to have the ability to make these changes.

Seasonal Haul

The concept of seasonal haul was developed in an effort to recognize that some forest roads are only open for use during certain times of the year. An example would be log haul during the summer months only, with the same road being closed during spring thaw. Therefore, designing such a road for the strongest time of the year instead of the weakest would greatly decrease the aggregate thickness required. Currently, only a few design methods explicitly consider this.

Field Experience

Field experience is simply a question of whether each design method has actually been used for design in the field. However, it should be noted that some procedures such as the AASHTO (*8*) guides receive such wide distribution all over the United States that it is not possible to determine if they have been used in design. The answers to this question are therefore only accurate to the best of the authors' knowledge.

Rating Scheme

Table 5 is a summary of all the aggregate surfacing design procedures included in this synthesis and their evaluation. Each of the nine attributes used for the evaluation have been discussed in the preceding paragraphs. The evaluation procedure was deliberately kept simple; therefore, weights were not assigned to each attribute. If a design method contained a particular attribute, it received a plus and if it did not, it received a minus. For those that fall within the grey zone between a yes and no answer, a zero is given. The pluses and minuses are then added together to arrive at a total score.

As can be seen from Table 5, two procedures have the highest score of +2: the Corps 1978 work by Barber et al. (*5*) and the Region 8 ARMS program. Note that the procedure by Barber et al. has more pluses than the ARMS program, although it also has more minuses. The Technical Advisory Board's opinion was that the pluses outweighed the minuses; therefore, the equation by Barber et al. was selected over ARMS.

Conclusions

To recapitulate, the direction for this project was to select the best available design method for use on forest roads. As noted, virtually all the design procedures reviewed, including that judged best by the Technical Advisory Board, have not been field verified. Field trials will be needed to validate the selected procedures, regardless of which procedure has been selected.

Earth Roads

At one time it was anticipated that Hammitt's (*6*) 1970 equation (Equation 4) relating thickness to coverages and wheel load would be most appropriate for the earth road design. However, the most important information sought by the designer is a prediction of either maintenance requirements or environmental damage resulting from allowing vehicles to operate directly on the native materials. Hammitt's 1970 equa-

TABLE 5 EVALUATION OF DESIGN METHODS

ATTRIBUTES	COE - 1988 TM 5-822-3 (7)	FHWA (C) COE - 1970 (Hammitt) (6)	Region 4 COE - 1978 (Barber et al.) (5)	R - 8 ARMS	SDMS - Manual AASHTO - 1986 FHWA (A) (8)	SDMS Computer	USFS Chap. 50 (1)	Region 1 Willamette	FHWA (B) (AASHTO 1972) (10)
1a. Validity for Aggregate Roads	+	-	+	0	0	-	-	-	-
1b. Validity For Earth-Roads	-	+	+	-	-	-	-	-	-
2. Inputs Make Sense	+	+	+	0	0	-	+	+	+
3. Standard Traffic Units	-	-	-	+	+	+	+	+	+
4. Varying Tire Pressure	+	+	+	+	-	-	-	-	-
5. Material Characterization	+	+	+	+	+	-	0	0	+
6. Risk/Reliability	-	-	-	-	-	+	-	-	-
7. Change failure criteria	-	-	+	+	+	+	-	-	-
8. Seasonal Haul	-	-	-	-	-	+	-	+	-
9. Field Experience	-	-	-	+	0	-	+	+	-
SCORE	-2	-2	+2	+2	-1	-2	-3	-1	-4

tion simply did not provide the means to estimate these maintenance needs or resource impacts. Consequently, the equation by Barber et al. (5) (Equation 5) was selected to predict total rut depth for the earth road situation, with some modifications.

Aggregate-Surfaced Roads

On the basis of the evaluation performed and after much discussion with the Technical Advisory Board, the equation by Barber et al. (5) was selected as the thickness design algorithm (Equation 5). It is recognized that the algorithm has some rather serious limitations. Perhaps the most serious is that the algorithm has little, if any, field experience. However, this is also true for most of the design algorithms reviewed. Equation 5 was developed at WES through a review of previous field data. The intent during the development was to provide a relationship as a starting point that could be refined through field experiments and experience in use.

A second major reservation that surfaced as a result of the evaluation was that the design algorithm appeared to underpredict thickness requirements for low-strength subgrade situations that might simulate wet weather haul. In spite of the shortcomings and reservations, this design algorithm was selected for the following reasons:

1. The design algorithm contains most of the design factors believed to be most important by the Forest Service.
2. The equation is stable with respect to the range of design inputs selected for use. The SDMS equation was unstable for some of the ranges of input values.
3. The algorithm was the most sensitive to changes in tire pressure, which was an input criterion. In addition, the thicknesses calculated were acceptable.
4. The design algorithm provided significantly reduced thickness requirements from those estimated through Chapter 50 for similar design inputs. This was consistent with the general perception that the Chapter 50 design method for aggregate surfaces is conservative.

In summary, there were clear reservations regarding the selection of the design algorithm. Unfortunately, the search of the existing thickness algorithms provided no clear choice, and it is the opinion of the Technical Advisory Board that the choice made was the best for forest road situations, given the state of the existing technology.

RECOMMENDATIONS

On the basis of the literature review and the analysis of the design methods in the preceding sections, the following recommendations are made:

1. For aggregate-surfaced and earth roads, the design algorithm selected was the model by Barber et al. (5) for aggregate-surfaced roads (Equation 5). The equation was slightly modified for use in the case of earth roads (the thickness of the compacted subgrade was assumed to be 6 in.).
2. One of the major disadvantages of many of the design procedures studied was their lack of field validation, particularly for forest road situations. Much of the test data that was used to develop many of the procedures was, as discussed earlier, from work done by the Corps in non-forest-related projects. In the development of SDMS, the Forest Service recognized this, and attempts were made to rectify this situation. It was anticipated that a data base from actual forest conditions could be used to validate or develop performance models for design of aggregate-surfaced and earth roads. However, for various reasons (e.g., insufficient traffic data) this was not successful. In the case of the Corps, projects to collect test data were also planned in cooperation with the Forest Service (5). However, they have not been followed up or the projects have been cancelled.

Therefore, the need for such field studies persists, regardless of whatever design method is selected. It is highly recommended that an attempt be made to gather some data that could be used to validate the selected design method and to enable future fine-tuning. Recognizing that data collection is expensive in time, money, and effort, it is suggested that good

use be made of any existing studies with related data, such as the Central Tire Inflation project. In addition, data collection has to be flexible enough to accommodate changes in test sites. Finally, an attempt should be made to collect only data that are needed. This should include traffic; loads; subgrade and cover material strengths over seasons, particularly wet seasons; and the monitoring of the selected failure criterion. The effects of moisture on subgrade strengths are particularly important.

ACKNOWLEDGMENTS

The research described in this paper was carried out by ARE Inc., Engineering Consultants, as part of a USDA Forest Service contract for the Region 6 office in Portland, Oregon. The authors are grateful for the support of the USFS and, in particular, Ron Williamson and the other members of the Technical Advisory Board.

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Pavement Design and Construction Specifications Developed for Low-Cost, Low-Volume Roads in Kenya

J. H. G. WAMBURA, M. S. ODERA, AND D. R. IKINDU

Appropriate standards have been developed for low-volume roads in Kenya. In the post-independence era, the road network expanded considerably and there was rapid depletion of high-quality materials both for maintenance and for new construction. The network had also expanded into difficult areas, devoid of high-quality road building materials but of high agricultural and economic potentials. Therefore the Low-Cost Pavement Research Program was started in 1982 with the aim of optimizing the use of local materials of marginal quality for pavement layers and thereby reducing total road costs. Particular emphasis was put on using laterites, quartzites, coral gravels, and weathered rock, on subbase, base, and surface layers. Different bituminous binders—straight-run bitumen, short residue, cut backs, and emulsions—were also studied. In order to achieve optimum results, the program was targeted to areas devoid of high-quality materials for pavement and surface layers. Several trial sections were located on various low-volume roads throughout the country.

In 1982, when the Low-Cost Pavement Research Program began, classified roads in Kenya totaled 53 000 km. Only 10 percent were bitumen surfaced; the rest were either gravel or earth surfaced. The network has now expanded to 62 000 km (*1*) but still with similar 1:9 ratio of bitumen to gravel or earth roads. The majority of gravel or earth roads falls within secondary, minor, and special-purpose categories presented in Table 1. Most of these roads have less than 100 average annual daily traffic (AADT).

Because the main objective of the program was to provide specifications for the use of local but marginally substandard construction materials on low-volume roads, investigations concentrated on the following areas (see Figure 1):

1. Kiambu—phonolites and laterites,
2. Kisii/Oyugis—weathered siltstones and laterites,
3. Majengo—quartzites and laterites,
4. Kwale—coral stones and laterites,
5. Lodwar—quartzites and weathered lava,
6. Narok—quartzites, and
7. Garissa—quartzites and kunkar limestones.

CHARACTERISTICS OF TRIAL SECTIONS

Selection and Location of Trial Sections

The trial sections for the program were located in different parts of the country to represent different climatic, geological, and subgrade conditions. Most of the sections were located on level and straight stretches of the road to obtain uniform drainage conditions on both sides of the section. Detailed construction of the trial section was in checkerboard formation. In this manner, the effects of directional traffic volumes could be observed.

A total of seven areas and 30-km length of trial sections selected are presented both in Figure 1 and Table 2. There were also two roads—A1 Marich–Lodwar (195 km) and D348 Lodwar–Kalokol (60 km)—which were constructed between 1978 and 1985 with similar substandard materials.

Ministry of Public Works, Materials Testing and Research Department, P.O. Box 11873, Nairobi, Kenya.

PAVEMENT LAYER CHARACTERISTICS

Construction Methods

Similar to the normal road construction projects, standard equipment was used to haul, mix, compact, and place pavement layers on the program. This included motor graders, power shovels, dump trucks, pulvimixers, rollers, water bowsers, bitumen distributors, and aggregate spreaders. However, special power screens were used to remove fines from aggregates for the gravel seal.

Compaction requirements for subgrade and subbase or base layers were 95 percent of British Standard (BS) proctor and AASHTO-modified standard, respectively.

Parameters for the cross section varied according to the class of the road. Carriageway and shoulder widths were 5 to 7 m and 0.5 to 2.0 m, respectively, with narrower widths on the earth or gravel roads and wider widths on new alignments and primary or secondary roads. To improve the drainage, side drains were lowered to minimum 0.45 m below the finished road level. Details of The D291 Majengo trial section are shown in Figure 2.

Subgrade Soils

Subgrade soils in the areas covered by the program consisted mainly of friable clays in wet areas and clayey sands in dry areas. The strength of subgrade soils varied between 2 and 15 percent California bearing ratio (CBR). A layer of improved subgrade (lower subbase) was augmented on sections with less than 7 percent CBR. Both subgrade layers were applied on new alignments of Sites 5, 6, and 7—Turkana, Narok, and Garissa, respectively.

TABLE 1 LENGTH OF CLASSIFIED ROAD NETWORK AS OF 1989 (*1*)

Class of road		Surface type			TOTAL
		Bitumen	Gravel	Earth	
Internat. Trunk Rds.	A	2,607.9	644.3	326.7	3,578.9
National Trunk Roads	B	1,171.2	928.3	641.0	2,740.5
Primary Roads	C	2,242.9	3,255.0	2,279.7	7,777.6
Secondary Roads	D	968.6	6,134.6	3,892.9	10,996.1
Minor & Sp. Roads					
Minor Roads	E	512.0	6,119.9	19,142.8	25,774.7
Government Access Rds	G	138.2	191.5	127.5	457.2
Settlement Roads	L	0.0	392.5	549.6	942.1
Rural Access Rds	R	14.7	6,972.7	729.7	7,717.1
Sugar Roads	S	6.7	80.0	858.1	944.8
Tea Roads	T	24.6	321.2	90.8	436.6
Wheat Roads	W	0.0	226.2	95.9	322.1
Total Minor + SP Roads		696.2	14,304.0	21,594.4	36,594.6
ALL CLASSES		7,686.8	25,266.2	28,734.7	61,687.7

Subbase and Base

The materials considered for these layers were laterites, quartzites, coral gravels, and weathered rocks. It was difficult to obtain gravels that met minimum requirements for low-standard bitumen-surfaced roads (*2*) of CBR 25 and 50 percent for subbase and base, respectively. There are additional requirements for plasticity index and proportion passing 0.075-mm sieve to be less than 15 and 35 percent, respectively.

These specifications, compared with those for high-volume roads, are relaxed. However, in most cases marginal strength properties are obtained even after stabilizing the materials with either lime or cement. It was therefore decided to scarify the existing base and add gravel to increase layer thickness from 150 to 200 mm on all sections with 20 to 30 percent CBR values.

INVESTIGATIONS ON SURFACE DRESSING MATERIALS

Scope of Investigations

Investigations on surface dressing materials were carried out in two main areas—aggregates and bituminous binders. The current specifications (*1*) do not allow stones with greater than 35 percent Los Angeles abrasion (LAA) and 26 percent aggregate crushing value (ACV) to be used in surface dressing. However, high-quality stones are scarce and not uniformly distributed within the country. Therefore, the use of alternative local materials (weathered stones and gravels) proved a viable option.

Unlike the aggregates, bituminous binders are more affected by the tropical climate. Generally, in a hot and dry climate

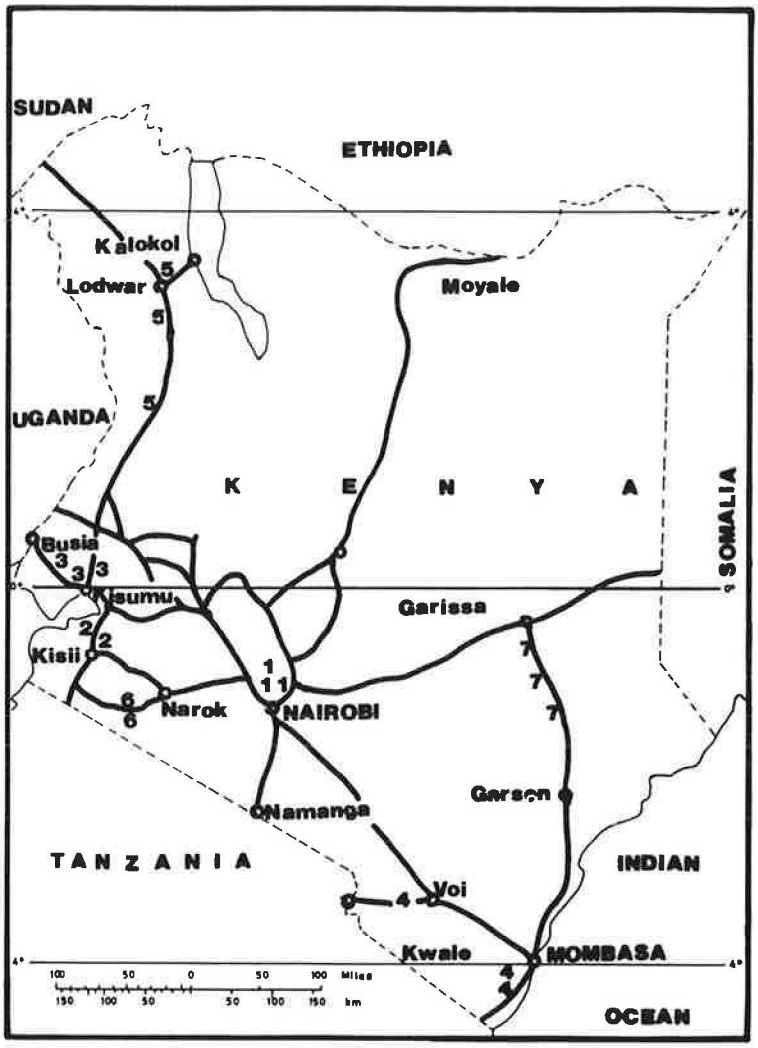

FIGURE 1 Location of low-cost trial sections.

the process of oxidation in binders is accelerated—the binder becomes brittle, then the surface cracks and deteriorates. This phenomenon is considered as one of causes of failures on the thin bituminous surfacings.

With the 80/100-penetration (pen.) bitumen as control, types of bitumen binders studied were as follows:

1. MC 3000 cutback blended from 80/100-pen. bitumen, diesel (fuel oil), and kerosene;
2. K1–60 and K1–70 cationic emulsions;
3. 250/500-pen. short-residue bitumen from Mombasa refinery; and
4. MB 2500 soft bitumen from Norway.

In order to increase adhesion, about 1 percent by weight of the antistripping agent diamin was added to the bitumen in liquid form.

Aggregate Properties

Important aggregate properties considered for the surface dressing trials were LAA, ACV, sodium sulfate soundness (SSS), bitumen affinity tests, and grading. Supplementary tests of water absorption and ACV on wet samples were carried out to determine the effect of water. Aggregate properties are presented in Table 3.

Except for the standard aggregates on the control sections that were applied in single nominal sizes, the weathered rocks and gravels were placed in a continuous graded matrix referred to as "Otta surfacing" (*3*,*4*) for both first and second seals. The continuous grading has major economic advantage of utilizing most sizes of the aggregates and hence minimizing wastage of gravel except for sizes outside 6 and 20 mm. The sizes of the continuous graded matrix were 6 to 20 mm and 6 to 16 mm for first and second seal, respectively.

In order to remove dust and improve on binder coating of the aggregates, particles finer than 6 mm were removed by use of power screens; wind and water cleaning of aggregates was necessary whenever available.

Spread and Spray Rates

The rate of spread of the aggregates varied between 70 and 100 m^2/m^3 from first to second seal. Because of both the

TABLE 2 CHARACTERISTICS OF TRIAL SECTIONS

Trial Section	Altitude (m)	Rainfall mm	Subgrade	Length km
1.1 C65 Ruiru	1540	1250	red clay	7.100
1.2 E409 Ndumberi	1540	1500	red clay	1.000
1.3 E437 Kiratina	1540	1500	red clay	0.650
1.4 AC Thuita	1540	1500	red clay	0.750
2.1. C21 Kisii	1530	2000	red clay	1.000
2.2. D220 Oyugis	1400	1610	red clay	1.000
3.1. D291 Majengo	1620	1790	red clay	1.000
3.2. E290 Bukuga	1620	1790	red clay	0.560
3.3. D264 Kima	1580	1790	red clay	0.560
4.1. C106 Kwale	90	820	brown sands	0.800
4.2. A23 Voi	650	500	brown sands	0.500
4.3. A14 Waa	20	1250	sandy soil	1.000
5.1. A1 Marich	1100	1000	sandy clay	2.600
5.2. A1 Lodwar	510	170	clayey sands	4.200
5.3 D348 Kalokol	460	170	loamy sands	0.320
6.1. C12 Narok	1900	820	brown clays	1.300
7.1. B8 Garissa	130	300	clayey sands	5.000

continuous grading and the rounded shape of the gravel aggregates, higher bitumen spray rates were necessary. The residual bitumen spray rates ranged from 1.0 to 1.4 L/m^2 depending on the size, shape, and cleanliness of the aggregates. Comparatively, the design manual's (2) residual bitumen spray rates for 10- to 14-mm nominal-size high-quality aggregates range from 0.8 to 1.2 L/m^2.

TECHNICAL EVALUATION OF THE TRIAL SECTIONS

Monitoring Program

After the construction of the trial sections was completed in 1985, a monitoring program was set to evaluate the performance of the sections. The monitoring program was carried out between 1985 and 1988, twice annually—after heavy rains from May to July and during the dry season of September to November. This procedure allowed the monitoring of the weakest and strongest states of the pavement. The parameters considered when evaluating the performance of the trial sections were as follows:

- Traffic census;
- Benkelman beam deflection measurements;
- Dynamic cone penetration (DCP) resistance;
- Visual inspections of drainage conditions and pavement defects (crackings, deformations, potholes, and patchings);
- In situ densities, moisture content, and gradings; and
- Roughness measurements from a towed bump integrator.

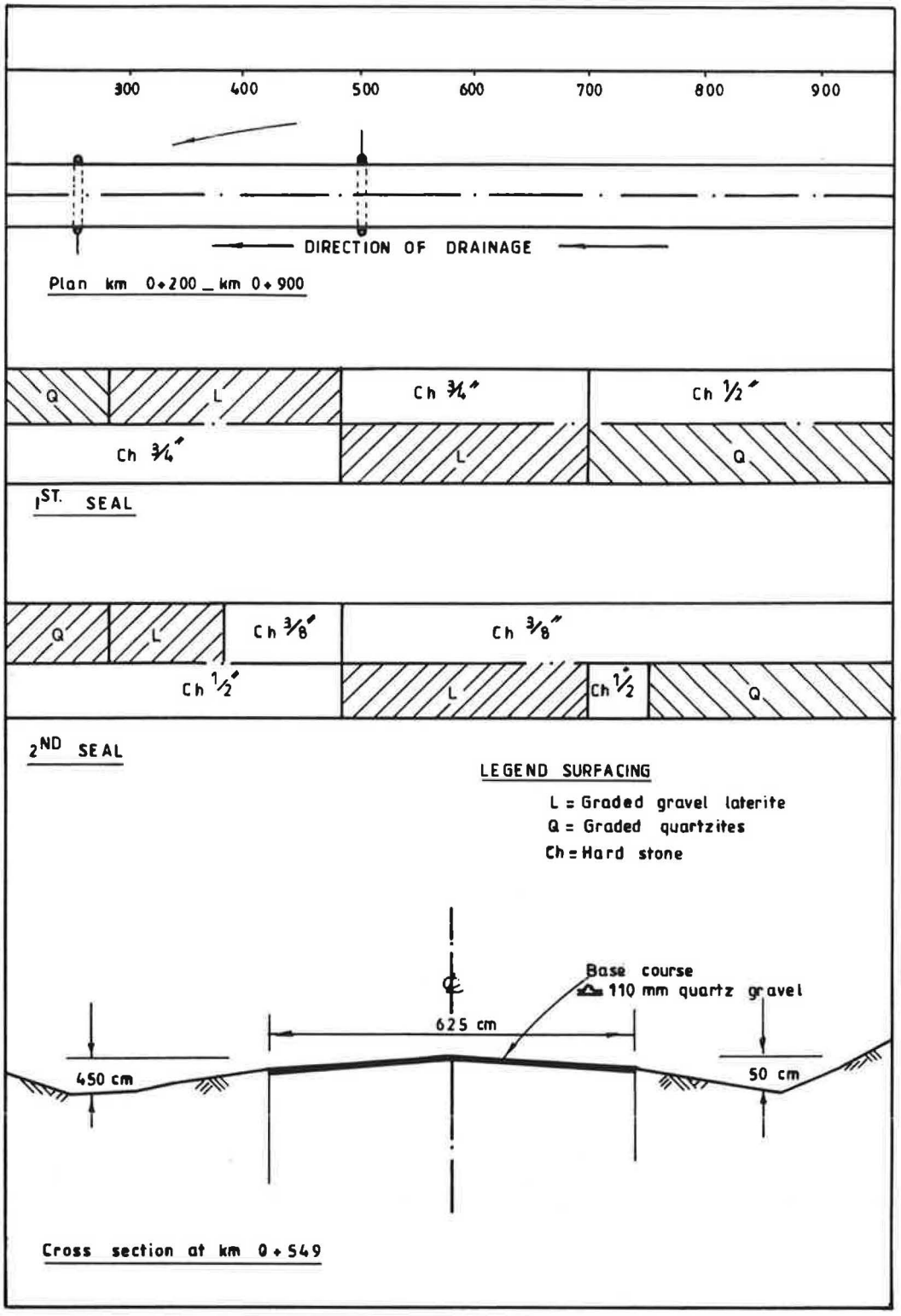

FIGURE 2 Details of D-291 Majengo trial section.

TABLE 3 PROPERTIES OF AGGREGATES ON THE TRIAL SECTIONS

Material	Abrasion test LAA[1] %	Crushing test ACV[2] dry %	Crushing test ACV[2] wet %	Chemical test SSS[3] %	Water absorption %
Ruiru laterites	70–83	48–55	59–62	61	11–13
Oyugis laterites	64	55	62	46	10
Kima laterites	73	48	57	90	9
Kwale laterites	37–67	45	51	9	-
Bukuga quartz	54	29	30	7	4
Lodwar quartz	45–49	26–30	-	-	-
Narok quartz	39	23	-	-	-
Garissa calcrete	29–40	23–30	-	-	-
Kwale/Waa coral	37	37	-	12	-
Ruiru soft tuff	44–51	32–37	48–56	26	16–23
Kisii siltstones	60	36	52	7	-
Voi (Taru grit)	37	28	-	3	-
Lodwar lava	49	32	-	-	-

Note: - indicates test results not available

1 LAA Los Angeles Abrasion

2 ACV Aggregate Crushing Value

3 SSS Sodium Sulphate Soundness

The results and recommendations from the monitoring program were presented in an internal report (*5*). A summary of observations follows:

- Marked increase of traffic AADT was recorded in all trial sections, probably because of the improved nature of the surfacing.
- Good relationship between deflections and DCP resistance was recorded.
- The depth of side ditches and carriage width decreased because of siltation, regraveling, and reshaping of the shoulders.
- The in-situ moisture contents, especially on the shoulders, fluctuated with the seasons. However, the dry density increased corresponding to the fines (<0.075 mm) on the base. This resulted from postcompaction from traffic and breakage of coarser gravel particles.

Performance of Aggregates for Surface Dressing

In general, the performance of the aggregates depended greatly on the hardness resistance (LAA and ACV values). As expected, the control sections constructed of quality aggregates provided the best performances. Next in descending order of performance were quartzites, weathered lava, siltstones, Kunkar limestones, coral stones, and laterites nodules. Apart from the laterites in Kwale, with reasonable LAA and ACV values, presented in Table 3, the laterites proved to be unsuitable as surface dressing aggregate for any traffic volume.

Performance of Bituminous Binders

The cutback MC 3000 exhibited the best performance, by holding the aggregates together and remaining live before becoming brittle and cracking. Next in performance, in the following order, were short residue (250 to 500 pen.), Norwegian MB 2500, K1–70 emulsion, straight run (80/100 pen.), and K1–60 emulsion.

Curing for cutback MC 3000, Norwegian MB 2500, and short residue took a long time (3 to 7 days), rendering them suitable only for new constructions but not for roads requiring immediate opening to traffic.

Antistripping agents are required to increase binder adhesion on the substandard aggregates and particularly on quartzitic gravels.

CONCLUSIONS

Overview

The results both from the Low-Cost Pavement Research Program and the Turkana road trials (*4*) since 1978 have provided valuable information to enable drafting of specifications for low-standard pavement. Further monitoring of the performance of the sections is continuing.

Despite lowered standards to accommodate local gravels for low-volume roads, materials strength, specification for LAA, ACV, and grading are still vital.

It was necessary to separate low traffic [Class T5 in the *Road Design Manual* (*2*)] into two categories, according to AADT and cumulative equivalent standard axles (ESAs), as follows:

- Very light traffic, 0 to 50 AADT and 0 to 100,000 cumulative ESAs; and
- Light traffic, 50 to 100 AADT and 100,000 to 500,000 cumulative ESAs.

Low-Standard Surfacing Aggregates

From the performances of the trial sections, it is recommended that laterites not be used as aggregates in surface dressing. However, quartzites, corals, calcrete, kunkar limestones, weathered lava, and siltstones can be used.

Both LAA and ACV are important strength parameters in the selection of surface dressing materials.

Grading is important in providing a stable interlocking matrix; however, removing aggregates finer than 6 mm by screening is recommended.

In order to enhance binder adhesion on aggregates, cleaning by water or wind is necessary, and for aggregates with poor bitumen affinity the use of antistripping agents is recommended.

TABLE 4 MATERIALS RECOMMENDATIONS FOR BASES

Parameters	very light traffic	light traffic	Requirements in Chapter 12 of the Manual (2)
maximum size mm.	10-40	10-40	10-40
fines 0.075 mm. %	< 40	< 35	< 35
plasticity index max			
dry areas[1]	< 25	< 20	< 20
wet areas	< 20	< 15	< 15
CBR (4day soak)min			
dry areas	25	35	50
wet areas	35	50	50
LAA max	70	60	-[2]
ACV max	45	40	-
thickness min. mm.	200	200	150

note - 1 dry areas have less than 800mm. annual rainfall

2 was not applicable

Guidelines for aggregates specifications for low-standard surfacings are as follows:

	Very Light Traffic	*Light Traffic*
LAA max (%)	60	50
ACV max (%)	40	35

Low-Standard Bases

The materials recommendations for bases are shown in Table 4.

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Design and Performance of Sprayed Seal Coats for Unbound Granular Pavements Carrying Heavy Logging Trucks

BRYAN D. PIDWERBESKY AND JOHN S. POLLARD

Virtually all of New Zealand's highway pavements usually consist of one or more coats of a sprayed seal over unbound granular layers. The standard New Zealand design and construction techniques are being applied to heavy-duty forestry roads. In a recent case study, a private forestry arterial road carrying heavily loaded logging trucks was examined. The advantages and limitations of New Zealand's approach to seal coat design with respect to the forestry roads are discussed. Bitumen application rates calculated from national highway design algorithms resulted in flushing in the arterial forestry roads because the surface treatment contains excess bitumen for the axle loads. Experimental work that has been instigated to further the development of seal coat design, construction techniques, and specialized equipment suited to the special requirements of forestry roads sustaining heavy loads is discussed. The concept of cover aggregate embedment and surface hardness is also considered.

Forestry companies are constructing and maintaining networks of high-quality private arterial roads, which are justified economically because logging trucks traveling at speeds of 80 to 100 km/hr over a smooth, all-weather surfaced road can haul logs at a lower cost per unit payload than would the railways. The logging industry takes advantage of New Zealand's unique highway pavement design techniques and practices, but some aspects need to be investigated and modified to suit the needs of the forestry roads. The background to the surfacing design techniques and practices is discussed first, followed by a case study.

BACKGROUND

The total population of New Zealand is 3.4 million. The majority live in five major cities, only one of which is inland. More scattered are rural towns supported by agricultural and pastoral industries that are still New Zealand's largest exporters. The road system that has evolved to serve these communities consists of long, sparsely trafficked lengths. Asphalt-bound aggregate systems are used for some urban streets and interurban motorways, and some rigid pavements were constructed 50 years ago, but virtually all highway traffic is carried by sophisticated unbound granular pavements. The main links between the population centers are classified as national highways and are funded by fuel taxes and heavy-vehicle road user charges. The national highways are managed by a national highway agency, Transit New Zealand [before 1989 the agency's title was the National Roads Board (NRB)]. The pavement engineering design and construction practices have been described elsewhere (*1–4*); the seal coat design, performance, and construction practices are discussed herein.

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Current Techniques for Designing Seal Coats

In New Zealand, surface treatments are called seal coats or chip seals; common types of seal coats are shown in Figure 1. The functions of the seal coat are to provide an impermeable membrane over the base course and a skid-resistant surface, as well as a wearing surface. The design of New Zealand's seal coats is based on the theory and mechanisms proposed by Hanson (*5*), who related the bitumen application rate to the size of the stone chip, the ratio of the chip's average, least, and greatest dimensions, and the residual void space within the single-layer thickness of the aggregate cover. Later, the major assumptions were refined by McLeod (*6*):

1. When one-size cover aggregate is spread over a bitumen film, the particles lie in unarranged positions and the voids between the particles are approximately 50 percent.
2. Rolling partially reorients the aggregate particles and reduces the voids to about 30 percent.
3. Finally, after considerable traffic, the particles become oriented into their densest positions, with all lying on their flattest sides, and the voids are reduced to approximately 20 percent.
4. Because the particles lie on their flattest sides, the average thickness of a surface treatment is the average least dimension (ALD) of the stone chips, as shown in Figure 2.
5. For good performance, under the typical traffic volume of 500 to 1,000 vehicles per day, the quantity of asphalt binder used should fill about 70 percent of the voids.

In New Zealand, the basic precepts have been refined by experience into a semiempirical design procedure that provides corrections for existing surface texture and vehicle loading, culminating in the *Seal Design Manual* (*8*). The seal design algorithm is shown in Figure 3. The algorithm is based on observations and studies involving public highways carrying normal traffic, which typically consists of 10 to 15 percent heavy commercial vehicles with an average axle load of about 5 tonnes. The surface texture is quantified by the sand circle test, which is the diameter achieved when 45 ml of sand (300

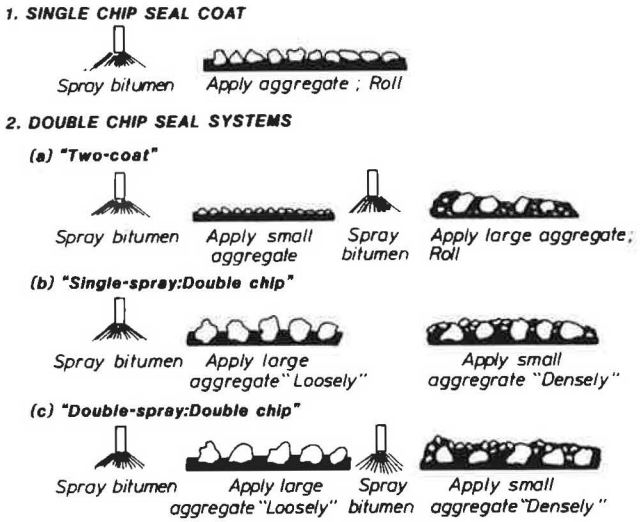

FIGURE 1 Types of seal coat systems (*7*).

to 600 μm) is spread by revolving a straightedge until the sand is level with the tops of the cover aggregate.

Factors Affecting Sprayed Seal Coats

Virtually all of New Zealand's bitumen is produced at the country's sole refinery, at Marsden Point, which uses crude petroleum from various sources to produce two standard penetration (pen.) grade bitumens—45/55 and 180/200 pen. A blend, 80/100 pen., is preferred in the warmer regions north of central North Island, and 180/200-pen. grade bitumen is used throughout the remainder of the country.

A rational basis for both modifying the bitumen with diesel and for temporarily softening it with kerosine was introduced in 1965 (*9*). Laboratory trials established the upper viscosity at which the various types of stone chip could still firmly adhere to a freshly sprayed bitumen film. The road surface temperature was assumed to be a function of the ambient air temperature and the percentage of cutback (usually kerosine) was adjusted accordingly to produce a target viscosity at the time of spraying.

Subsequently, information on the viscosity-temperature-cutback relationships for bitumens used in New Zealand has been extended, and modern instruments have enhanced measurement of the true surface temperature. Field measurements have shown that the relationship between air and road temperature is rather more complex (*10*). Nevertheless, the basic principles have remained the same.

In addition to material properties and environmental factors, seal coats are dependent on operator skills and equipment precision. Fortunately, under normal traffic loadings, errors in bitumen application arising from incorrect design assumptions, departures from theoretical binder formulation, irregularities in sprayer performance, or minor departures from specified practices tend to negate the effects of each other. Moreover, a typical seal coat subjected to common loading conditions has considerable inherent tolerance. Adhesion agents are usually added to the bitumen.

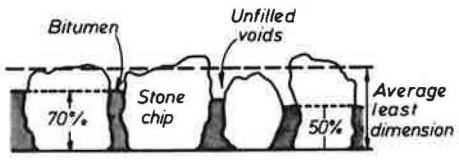

FIGURE 2 Cross section of a seal coat system.

Generally, there have not been serious problems on most public roads, though occasionally excess bitumen flushing from old seals has had to be burned off. However, beginning in the early 1970s, an increase in the occurrence of flushing in pavements under increasing traffic volumes led to a growing awareness of sprayer maladjustments. The main causes were incorrect bar heights and worn, misaligned slot jets (slot jets predominate in New Zealand, both for cutback and emulsion spraying). *The Performance of Bitumen Distributors* (*11*) was introduced to ensure an application rate precision in the order of ±2.5 percent. Subsequent testing of sprayers indicated that some did not have the precision essential for producing uniform-bitumen films at low application rates, principally because the sprayers had not been designed for such duties.

The cover aggregate used in New Zealand seal coats is always crushed-stone particles of uniform size, even though this is more expensive than a graded-cover aggregate. Particle size range and shape has been tightly specified and controlled for many years, so that a good mosaic is produced in the seal coat cover aggregate. The design procedure assumes that the void volume is still 20 percent, though modern stone-crushing plants produce a more cubic chip than the norm of 50 years ago.

Until recently, it was also believed that heavy rollers were essential to chip embedment but this apparently self-evident

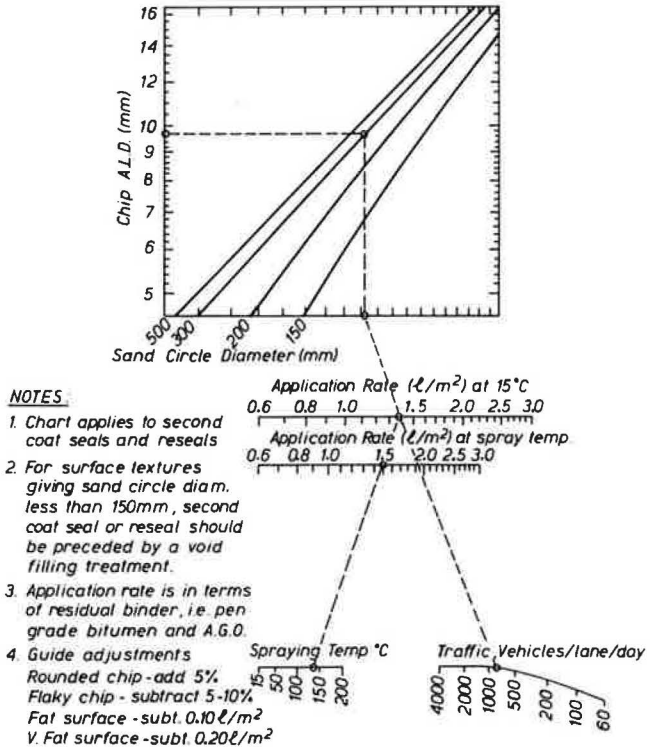

FIGURE 3 New Zealand seal coat design algorithm.

premise has been disproved (*12*). The research indicates that the mass of the roller compactor is less important in creating a tightly locked mosaic of the stone chips than tire action. The research also showed that excess cover aggregate interferes with particle placement and early alignment under trafficking, both of which are essential for proper embedment at low bitumen contents.

In spite of the theoretically rigid requirements, it is not uncommon practice in New Zealand for contractors on private work to exercise an appreciable degree of experience-based judgment in determining the appropriate bitumen and aggregate application rates for specific situations. The application rates of bitumen tend to be higher to avoid risking loss of stone chip. As a precaution against loss of chips by traffic action, the actual application rates of cover aggregate also tend to be higher than the rates derived from theoretical design procedures.

Vehicle Loading

The tradition of a strong central government has resulted in uniformity of pavement design and construction as well as a concomitant enforcement of heavy vehicle load limits. At present, the vehicle configuration is limited to a total maximum length of 20 m for an A- or B-train, hauling no more than one trailer behind a tractor-semitrailer combination. The maximum weight for the vehicle is limited to 44 tonnes, and the maximum loads permitted on single, tandem, and triaxle groups are 8.2, 15.0, and 18.0 tonnes, respectively.

Heavy-vehicle loading is quantified for highway pavement design purposes in terms of equivalent design axles (EDAs). The NRB reference loads for single- and dual-tired axles are 6.7 and 8.2 tonnes, respectively; the standard tire pressure is 580 kPa. Actual axle loads are related to the reference loads by the fourth-power rule (the exponent is 4.0). Currently, transport operators are lobbying to have the load limits relaxed, but the existing data base is inadequate and pavement engineers are reluctant to extrapolate the present pavement design techniques for untested loadings.

In recent years, increasing lengths of private roads have been built by logging companies in the radiata pine forest plantations of New Zealand. The heavy haulage vehicles for which these roads are built are not subject to the load limits imposed on the national highway systems and some seal coats over unbound granular pavements are subjected to axle loads of up to twice the current national limits. The following case study illustrates how inappropriate the semiempirical national highway design approaches are for seal coats subjected to such loads. Conversely, empirical data derived from such conditions could be used to enhance the national highway design procedures.

CASE STUDY

The case study involves a 50-km forestry road constructed in the scenic northeastern region of the North Island. In 1987, a major forestry company calculated that private truck haulage would be more economic than public rail haulage to transport logs between a major assembly depot and its pulp and paper mill (the largest in the southern hemisphere).

The thicknesses of the pavement layers in the arterial road were designed using the state highway pavement design manual (*4*); the selected design life was 15 years. Most of the road consists of a 200-mm-thick granular base course and 100-mm-thick granular subbase over a subgrade stabilized with lime or soil cement. The first coat seal consisted of 180/200-pen. grade bitumen, cutback with 7 percent kerosine, and cover aggregate of ALD 5.5 to 8 mm. The seal coat was applied in stages during the period December 1986 to March 1988. The application rate of the bitumen (at 15°C) ranged between 1.15 and 1.24 L/m^2. All materials and construction practices followed national highway specifications (*13–18*).

One year later, in accord with normal national highway practice for the region, the road received a second seal coat. The bitumen was 180/200-pen. grade, cutback with 3 to 4 percent kerosine. The application rate of the bitumen (at 15°C) ranged between 1.97 and 2.36 L/m^2, depending on the surface condition. The aggregate was a larger size of stone chip of ALD 9.5 to 12 mm.

Less than 2 months after the second seal coat had been applied, bitumen in the wheel paths of the loaded lane had flushed to the extent that free bitumen was present on the surface. The stone chips were still in place and were not being removed by vehicle tires, except at intersections where severe turning was necessary. Surface excavations revealed that the second coat of larger particles was being pushed down into the lower layer of smaller particles. The first coat of cover aggregate and bitumen had apparently bonded well to the base course. The base course had a firm, distinct surface, which implied that the chips were not punching into the base course and that the bitumen was not being absorbed into the base. The base course surface was dense and well compacted, and appeared to have the normal moisture content of approximately 2 percent. The bitumen was mobile, which confirmed the absence of fine particles at the bottom of the seal coats. Patching was only necessary in the few places where the whole chip-seal system had been removed by a tire after a parked vehicle had moved away.

The lane carrying unloaded vehicles was flushing also but only to a minor degree. Apparently, the effects of weathering had kept pace with flushing so that no bitumen from a broken skin was apparent. The surface of untrafficked areas exhibited the locked mosaic of particles expected of a well-constructed seal coat.

Remedial Work

After the flushing started, a variety of remedies was attempted. The first trial involved spreading stone chips precoated with bitumen. This process is successful in some situations on public roads but was unsuccessful in this particular case, probably because the stone chips were too large and the seal coats did not need more bitumen. A second trial that involved applying thin layers of small aggregate (ALD of 3 mm) was unsuccessful because the truck tires threw them off. Burning off the excess bitumen was not attempted in the heavily forested area.

A more successful solution has been the application of a thicker layer of 3 mm (ALD) cover aggregate. The lower chips in the layer are pressed into the seal and stick to the binder while being protected by the covering chips. But, to determine the most suitable long-term solution, a compre-

hensive investigation was undertaken by a team of four final-year engineering students from the University of Canterbury.

Subsequent Investigation

During the summer period of November 1989 to February 1990, various aspects of the arterial forestry road were investigated in detail. Initially, construction records were reviewed and a detailed description of the road was compiled. A visual appraisal survey and photographic log of the surface condition were done for the whole road.

Benkelman beam tests under an 80-kN axle load were conducted in the inner and outer wheelpaths of both lanes every 50 m along the entire length of the road. Most of the road exhibited deflections of 0.5 to 1.2 mm, which is typical for New Zealand highways; about 10 percent of the road length had deflections of 1.2 to 1.9 mm, which is acceptable for public highways subjected to typical commercial vehicle loads.

Axle weights and gross weights of all the logging vehicles using the private forestry road were measured; a relationship between vehicle configuration and payload efficiency, with respect to potential damage to the road, was developed. The average number of fully laden vehicles traveling over the road was 140 per day. The maximum gross vehicle weights of the trucks ranged from 40 to 120 tonnes; the maximum axle loads were 15 tonnes per axle, in tandem or triaxle groups. Cold tire pressures ranged from 650 kPa on trailers to 730 kPa on truck driving axles (of Mack trucks).

The bearing capacity of the subgrade and the pavement and subsurface drainage were evaluated. The excavations revealed that the base course had a sound, hard surface, and was composed of quality aggregate.

Samples were taken of the seal coat from both flushed and nonflushed seal coat sections; extracted bitumen was tested for diluent content (see Table 1) and aggregate was examined for quantity and dimensions. The substantial diluent contents remaining after 2 or 3 years suggests that either, initially, the actual contents were higher than those recorded because of imprecision in the mixing or the diluent is remaining in the bitumen much longer than is normally assumed. The actual application rates of the bitumen and the cover aggregate deviated substantially from specified values. The contractors confirmed that application rates were adjusted on-the-spot on the basis of visual assessment of the road surface and experience.

TABLE 1 PROPERTIES OF RETRIEVED BITUMEN SAMPLES

Age of each seal coat		Diluent Content
First (years)	Second (years)	%
2	1	2.2[a]
3	2	2.2[b]
5	3	2.1[c]

[a] Average of 3 samples.
[b] Average of 6 samples.
[c] 1 sample.

A theoretical model describing the performance of bitumen in seal coats was also derived, and will be calibrated using actual field data. The final analysis and report were incomplete at the time of the writing, so specific details and references are unavailable.

The information is being compiled into a data base for easy retrieval and updating.

Discussion

The flushing, although severe, differed only in degree from that of normal seal coats made with an excess of bitumen. The prime cause of flushing was a seal coat that was inappropriate for such a major departure from the orthodox highway loadings, on which the normal seal design and construction procedures are based.

If the excess is small, and the rate of chip consolidation slow, then surface oxidation will keep pace with the flushing so that over the years the bitumen, although it becomes level with the top of the chips, never reaches the stage of open flushing. An inspection of the public highways in the same region showed that flushing occurred over most of the surface, but was not a problem.

In a research project conducted at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) in Christchurch, New Zealand, during 1987, pavements similar to the case study road were subjected to 40-kN dual-tired wheel loads at an average speed of 40 km/hr until 1.4 million loads had been applied. Soon after loading began, the initial two-coat seal began flushing, even though there was no loss of chip and the base course was firm. After flushing became severe, the initial coats were removed and the base course lightly releveled. A single coat of bitumen sprayed at a lower-than-normal rate was coated with a first layer of larger stone chips (ALD of 12 mm) interlocked with smaller chips (ALD of 8 mm). This seal coat began exhibiting severe flushing after only a few thousand vehicle loadings. The flushing was attributed to excess bitumen of too low a viscosity for the high axle loads, the high rate of loading, and the absence of light-vehicle traffic that could have slowly conditioned the surfacing (*19*).

The problem has also been noted in Australia. Oliver (*20*) reports that, although Australian bitumen quality has remained relatively constant, there are complaints that

- Seal coats that would not previously have bled in the wheelpaths are now doing so, and
- Bitumen remains lively for longer periods before setting-up, or months or years after construction it becomes lively in hot weather.

The possible causes are (*20*) the following:

- The advent of triaxle configurations. These tend to have a poor load distribution between the axles in the group. For example, 1 in 10 trucks surveyed had a load variation exceeding 20 percent on individual axles of the triaxle group.
- The adoption of wide single tires in place of dual-tired wheels. In the worst case, the load can approach 5 tonnes per tire.
- At the time of the AASHO Road Test, typical truck tire pressure in the United States was 560 kPa (*21*). Average tire pressures now range from 730 to 860 kPa.

Oliver (*20*) concludes that the degree of embedment of chips will depend on the numbers of and characteristics of the heavy vehicles, such as gross weight, suspension type, and tire characteristics, as well as the resistance of the underlying layer to embedment. The forces and mechanism are such that the properties of the bitumen will have negligible effect on the process. When embedment occurs, bitumen is forced to the surface and flushing occurs in the wheelpath. If a reseal is applied to correct the problem, then the reseal is likely to be affected in the same way.

For the arterial forestry roads, the axle loads are considerably in excess of the values on which the national highway design tables are based. The design input, such as ALD values of the particles, surface texture, and vehicle intensity, used to derive the bitumen application rates were based on NRB practice for orthodox vehicle loadings. Bitumen of 180/200-pen. grade was used for the second-coat seals in the belief that the bitumen normally used in the region (80/100-pen. grade) would be too brittle in the winter. (NRB experience indicates that cracking does not occur provided the pavement deflection is of the shallow-shaped bowl variety, and the minimum temperatures (-8°C) are not low enough to cause cracking.)

Another contributing factor may be that the arterial forestry road had to carry the full working load immediately following construction. Older forestry pavements in the same region, whose traffic loadings with respect both to axle numbers and load magnitude have increased at a lower rate over many years, have performed well.

The geometrics of the arterial forestry road are particularly good. This factor enhances the efficiency of the trucking operation but, as a consequence, heavily loaded vehicles of similar configurations travel steadily at speeds in excess of 80 km/hr along a common wheelpath as undeviating as a rail line. The intensity of the wheelpath use is much greater than that of a public highway where overtaking, varying vehicle dimensions and tire spacings, and driver behavior provide random deviation of the wheelpaths, yielding a broader transverse distribution.

REVIEW OF OTHER SEAL COAT DESIGN METHODS

The Transit New Zealand method is based on normal highway traffic; axle loads are limited to 8.2 tonnes per single axle carrying dual-tire wheels, and the maximum allowable tire pressure is 825 kPa for radials. A review of international literature was undertaken to determine if any other surface treatment design techniques may have been developed that do consider heavy-axle loads, tire characteristics, and surface hardness.

Houghton (*22*) evaluates a variety of New Zealand and French design techniques to derive a suitable method for heavily trafficked urban arterials, and Kandhal (*23*) presents a comprehensive review of surface treatments, but none consider the three critical parameters listed earlier. Southern (*8*) and Road Note 39 (*24*) consider commercial vehicles of unladen weights over 1.5 tonnes and take into account a subjective description of the surface hardness of the road. Potter and Church (*25*) proposed that seal coat design should be based on (a) hardness (resistance to embedment) of the layer under the seal coat, and (b) traffic loading. Their paper provides the initial basis of such a design approach, but emphasizing that long-term data are still required to evaluate the embedment of the cover aggregate in the underlying layer under trafficking. Also, the results were limited to first-coat seals using a 16-mm stone chip.

Seal coats are an attractive economic alternative for low-volume roads but none of the design procedures are appropriate to the loadings that are already being experienced on the arterial forestry roads.

PROPOSED DEVELOPMENTS

Seal Coat Design for Low-Volume Roads under Heavy Axle Loads

An important aspect deserving immediate attention is the matter of the exponent or relationship to be used in the load equivalency conversion factor. It is unlikely that the fourth-power law used on national highways is applicable to the forestry road loading conditions, especially in designing the seal coats.

The case study results confirm that the bitumen content must be kept low. The actual application rates in the case study were 1.15 to 1.24 L/m^2 for the first coat and 1.97 to 2.36 L/m^2 for the second coat. Using the NRB design algorithm and judgments of correction factors based on hindsight, the required bitumen application rate was calculated to be 0.8 L/m^2 (at 15°C) for the first coat seal. The application rate was also calculated by the Asphalt Institute method (*26*). Assuming that the base course was reasonably smooth and slightly porous, and the logging traffic applied tire loads equivalent to a normal public traffic mix of greater than 2,000 vehicles per day with about 10 percent heavy commercial vehicles, the first-coat application rate of residual bitumen was calculated to be 0.9 L/m^2 (at 15°C).

A computer-based expert system package, the Australian Road Research Board (ARRB) Sprayed Seal Advisor, was also used to calculate the required bitumen application rates:

- First coat of cold, residual bitumen of 0.8 L/m^2. (This value is equivalent to 0.86-L/m^2 cutback with 7 percent diluent.)
- Second coat of 1.2 L/m^2. (This value is equivalent to 1.25 L/m^2 cutback with 4 percent diluent.)

However, the theoretical values do not incorporate the necessary component of a visual appraisal of the condition of the surface before being sealed.

The feasibility of priming under arterial forestry road conditions should be examined. Emulsified bitumen could then be used even for the first seal coat. This would avoid the curing time of cutback diluents and wet the maximum particle surface area although the residual bitumen application is low.

A seal coat design that should be studied is a single-spray, double-chip seal incorporating a large chip braced and locked by a smaller top chip. With proper size selection, this design can produce a tight seal with a low total volume of bitumen. A double-spray, double-chip seal should also be considered, but the total bitumen content must be kept low.

These techniques are practical and available, and should be evaluated by field trials conducted on arterial forestry roads

in use, which would yield results after many years, following preliminary trials at the Canterbury accelerated pavement testing facility. The test track trials, with the capability of applying 15 years of trafficking in a few months, will enable the evaluation of seal coat designs outside normal prudent limits because of the lower cost of test track utilization.

Road Engineering and Management

In New Zealand, the low-volume roads used by the logging industry carry loadings considerably in excess of those currently allowed on public highways. If modern-road engineering management techniques had been applied to the forestry road system, the problems highlighted by the road studied could probably have been predicted. The forestry road operators need to build an ongoing data base of experience and experimentation. Substantial attention and resources should be dedicated to monitoring existing roads and documenting activities. The pavement management techniques used by New Zealand's public road authorities are unsuitable because of the difference of loadings and users' needs. No road information and management system is known that is suitable for the forestry road situation. The logging industry should also investigate the feasibility of providing financial incentives to haul operators to use less damaging axle load and tire configurations.

CONCLUSIONS

The current seal coat design procedures and construction practices are unsuitable for low-volume roads carrying heavy axle loadings.

The requirements of low-volume arterial forestry roads subjected to heavy axle loads are superior to those acceptable for New Zealand national highways. Thus, sealing practices, especially with respect to bitumen and stone chip application rates, additions of cutback diluent, and adhesion agent, require supervision and equipment able to readily comply with the more rigorous requirements.

It is essential that the operators of low-volume roads build their own data base from experience and experimentation. More specifically, an ongoing system of monitoring and documenting the planning, design, construction, performance, and maintenance of arterial forestry roads should be implemented.

The performance of a single-spray, double-chip or double-spray, double-chip emulsion seal over a primed surface under heavy axle loads should be investigated. Long-term field trials, complemented by testing in an accelerated pavement testing facility, are recommended.

ACKNOWLEDGMENTS

The authors are grateful to Tasman Forestry Limited for permission to publish information derived from their forestry road operations, to J. Oliver for his advice and comments, and to the summer research team of J. Hadley, W. Hatcher, T. Preston, and B. Steven for the information gathered during the arterial forestry road investigation project.

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Cross Sections and Pavement Subdrainage of Low-Volume Roads in Madagascar

José Carlos de O. S. Horta

Distress of poorly drained pavements typically takes the form of longitudinal settlements and cracks along shoulders (both shoulders of straight aligned stretches and the lower shoulder of superelevated curves) and results from water penetrating the structural layers of the pavement system, flowing along the transverse and longitudinal grades, and saturating materials of lower areas. Water flowing in the pavement along the longitudinal gradient may feed perched water tables in pavement layers of low embankment sections. Seepage from the perched water tables into the shoulders and subgrade may originate edge slides. Pavement cross-section design should take into account the permeability of available natural materials as well as processed base materials. Adequate provision for prompt outflow of pavement water should be given except for impervious pavements that do not require to be drained and for pavements over pervious subgrades where efficient subdrainage is achieved by percolation to deep-water tables. A catalog containing eight standard pavement cross sections has been proposed for the Malagasy low-volume, bitumen-paved roads and the unit prices of 10 road construction and rehabilitation projects were used to derive comparative construction costs of the different standard cross sections.

What is most important for low-volume road pavements: thickness design or subdrainage design? Both thickness and subdrainage design are essential for good performance of low-volume road pavements. However, thickness design is usually carefully studied and investigated whereas subdrainage design is not always contemplated.

Neglecting pavement subdrainage design results in reduction of pavement service life, but in many cases the role of poor drainage in pavement distress and failure goes unnoticed as inaccuracy of traffic assumptions may also account for shorter service lives.

From time to time, the occurrence of pavement failures obviously caused by poor subdrainage is warning road engineers of the importance of subdrainage design.

A CASE HISTORY

National Road 12 between Irondro and Manakara along the southeastern coast of Madagascar was designed for a service life of 15 years. During construction, the subgrade compaction and bearing capacity were carefully checked and the pavement thickness design adjusted to comply with the actual subgrade California bearing ratio (CBR) value. Construction was completed by the middle of 1983 (*1*).

The pavement comprised a subbase with variable thickness of natural sandy materials selected on the basis of a soaked CBR exceeding 30 percent, a 15-cm-thick crushed basalt base, and a double surface dressing. The base course was 40 cm wider than the surface dressing. The base course edges did not receive surface dressing and were simply primed. Subbase materials were used in the shoulders. The shoulders and subbase materials were either impervious clayey sands and fine soils or pervious, coarse river sands.

The very pervious crushed basalt base course extending beyond the wearing course provided conditions for the easy ingress of water in the pavement. Water outflow and pavement drainage were provided by transverse crushed stone drains located in the shoulders every 10 m as usual in Madagascar.

The eastern coast of Madagascar is exposed to a humid climate without a dry season. The average yearly rainfall in the region where the road was constructed is around 2400 mm. The so-called "summer," from November to April, is the season of tropical cyclones with heavy rainfall. During this season, the hourly precipitation may reach 60 mm or more. Rain may fall continuously for 3 days or more.

Before the end of the warranty period and during the heavy rainy season from November 1983 to March 1984, severe pavement distress including settlements, longitudinal cracks, alligator cracking, rutting, stripping, raveling, and potholes as well as embankment instability developed in many sections of the road (*2*). By this time, the traffic using the road was less than 50 vehicles per day. In many places (Figure 1), water was seen flowing from potholes and cracks after rain showers. These pavement springs were situated both in cut and embankment sections, in plains, as well as slopes and the tops of hills.

Distress only occurred in stretches where the shoulders and the subbase were constructed with impervious materials. Sections of pavement with pervious shoulders and subbase remained in good condition and did not show signs of distress even after the heavy rainy season. The only exception was a pothole and pavement spring that appeared soon after construction in the lower part of a horizontal and vertical curve. The pothole was obviously caused by water pressure build-up following infiltration in the superelevated upper shoulder and showed abundant water outflow.

The relationship between inadequate pavement drainage and premature failure was clear for National Road 12.

TYPICAL DISTRESS OF POORLY DRAINED PAVEMENTS

The ingress of water in road pavements should be avoided and limited by appropriate design and construction materials.

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FIGURE 1 Alligator cracking, loss of fines, and springs along the centerline of an embankment section at Kilometer 21 + 700, National Road 12.

The construction of watertight wearing courses may be achieved by using good materials and workmanship but this condition cannot be maintained for long periods of time. After a few years of service, joints, cracks, and relatively pervious areas will allow water to ingress into the pavement. When appropriate drainage is provided, the harmful effects of water in the pavement structure can be avoided.

In the absence of appropriate pavement subdrainage, water may seep into the pavement system and flow along the transverse grade to the pavement edges as well as along the longitudinal gradient to low sections. Where impervious shoulders oppose flow, seepage of water into the subbase and the subgrade results in a loss of bearing capacity and traffic loads produce longitudinal settlement and cracks along the shoulders (Figure 2). This combination of distress types is characteristic of poorly drained pavements. In straight alignments, water-related distress develops along both pavement edges but in curves it only develops along the lower shoulder.

Longitudinal flow of water in poorly drained pavement layers may feed perched water tables in low embankment sections. Seepage from perched water tables through the subgrade and the shoulders may cause edge slides.

POSSIBLE DIFFERENT WAYS OF DRAINING PAVEMENTS

Where the base course has low permeability (soil-cement, soil-lime, soil-bitumen) relative to the shoulders, the subbase, and the subgrade, pavement subdrainage is not critical.

If the base course materials are pervious, one possible way of draining the pavement is vertical infiltration or percolation to a deep water table through a pervious subgrade. Cohesionless sand or gravel may be pervious enough that specific pavement subdrainage is not required.

On impervious subgrades, another possible way of draining pavements is through pervious subbase and shoulders.

A third possibility is lateral drainage from the base to the shoulders. These may be constructed with pervious materials or made selectively pervious by means of shoulder transverse drains. The drain section and spacing should be properly designed (*3–6*).

A fourth possible way of draining pavements is by means of a special drainage layer or blanket at the base of the pavement with longitudinal perforated collector pipes at one or both edges and outlet pipes under the shoulders.

Transverse interceptor drains and herringbone drains may be useful under certain conditions. Herringbone drains are often used to improve drainage of sections where a drainage blanket was neglected during construction. For this purpose, trenches at a certain angle with the centerline are cut through both lanes and filled with draining materials.

Shoulder drains were traditionally used in Madagascar. Their performance is sensitive to construction conditions and workmanship. Problems occur if the drain trenches across the shoulders are cut before base course compaction. If the trenches are cut before completion of base course construction, the drain inlet may be disturbed and plugged by construction equipment. The shoulder drain should penetrate at least 10 cm into the pavement base course and the bottom of the drain should be lower than the bottom of the base course. As natural gravels are scarce in the Red Island, shoulder drains were traditionally constructed by backfilling the drain trench with crushed stone and priming the surface. This technology gives no proper security against penetration of fines, plugging by vegetation and animals, and damage by occasional traffic.

FIGURE 2 Longitudinal cracks and settlements along the shoulder, typical of poorly drained pavement.

FIGURE 3 Edge settlement and water outflow through longitudinal cracks.

Geotextile encapsulation appears to be a sound alternative for proper operation of shoulder drains and should be promoted.

Section design and spacing of shoulder drains should be based on the expected rainfall, the infiltration rate, and the permeability of the base course. The experience of National Road 12 showed that the usual 10-m spacing was conservative for the highly pervious crushed basalt base course in the conditions of the heavy rainfall season. Shorter spacings of 5 and 3 m are recommended in tropical countries (*7*), when a continuously pervious shoulder would be too costly.

CATALOG OF PAVEMENT STANDARD CROSS SECTIONS

The case of National Road 12 called attention to the utmost importance of subdrainage design. One of the main lessons to be learned from this experience is that standard pavement cross sections cannot be considered independently from local pavement materials, particularly the pavement materials permeabilities.

One efficient way to ensure pavement drainage is to extend the pavement materials across the shoulders. This is usually done for the subbase materials, which are low-cost natural materials, but not for the base materials. These are usually crushed stone in Madagascar. In order to decrease construction costs, natural materials, usually the same as the subbase materials, are used in the shoulders.

In Madagascar, most subbase and shoulder materials are impervious clayey sands. Turfing is applied to the surface of shoulders to protect against erosion. Gravels such as rolled river gravels and lateritic gravels are very scarce in the island. In some particular regions, soft limestone and volcanic gravels are available. Gravelly, coarse well-graded sands can be found along some rivers.

In order to avoid future mistakes in pavement cross-section design, a catalog of pavement standard cross sections (*8*) was prepared by the highway department in Madagascar.

This catalog did not specifically address the problem of water ingress from the subgrade into the pavement. Where required, this problem should receive particular treatment. The specific problem of the catalog was to provide drainage for water trapped in pervious, coarse pavement materials. Consequently, the problems of internal drainage and the requirements for filter layers were not developed.

The Malagasy catalog (Figure 4) comprises eight different cross sections to choose from for each pavement stretch in accordance with the available natural or processed pavement materials.

Cross Section 1: Impervious Base

For better lateral support, the impervious base (soil-cement, soil-lime, soil-bitumen, or waterbound macadam with impervious filling) is extended 20 to 30 cm laterally beyond the wearing course. Low-grade shoulder materials with turfing may be used. Cross Section 1 should not be used with pervious base materials such as crushed stone. The base edges without wearing course would allow the ingress of water with resulting distress, as discussed previously.

Cross Section 2: Full-Width Base

The pervious or impervious base is extended the full pavement width. The resulting cross section is costly but has many advantages: easy construction comparative to cross sections where base and shoulder materials are different, good lateral support, strong shoulders, and easy subdrainage for pervious base materials.

Crushed-stone shoulders are erodible by occasional traffic. To avoid shoulder erosion, surface protection can be contemplated. Simple priming is often suggested but does not appreciably improve surface resistance to erosion by traffic. Surface dressing and stone lining or paving are effective for this purpose and, in order to reduce costs, can be restricted to particularly exposed shoulder sections.

Cross Section 3: Pervious Base, Impervious Subbase, and Pervious Shoulders

The base course is not extended over the shoulders, but shoulder materials different from the subbase materials and more pervious than the base are used to ensure lateral permeability.

Construction cost is reduced relatively to Cross Section 2 because of low-cost shoulder materials such as crusher run or pervious natural gravels, but the latter are only available in a few regions of the Great Island.

Overlaying of an existing pavement with a pervious base or subbase would require previous reshaping of existing impervious wearing courses to avoid water ponding in the new base or subbase. The same result can be achieved at lower cost by total or selective scarification of the existing bituminous surfacing. Grooves excavated across the existing surfacing may ensure drainage of the overlay provided the existing pavement has suitable subdrainage.

Cross Section 4: Pervious Base and Subbase

The pavement is drained by the pervious subbase. Water may infiltrate through superelevated shoulders, seep towards the structural pavement layers, and cause waterhead build-up. To avoid this, the higher superelevated shoulder in curves should either be surfaced or constructed with impervious materials. Cross-section design with adequate subgrade transverse grading (Figure 4b) would also avoid or limit seepage of rain water toward the center of the pavement and along the longitudinal gradient to build up high pressure.

Cross Section 5: Pervious Subgrade and Subbase

In sandy areas, pavement subdrainage takes the form of percolation through the pervious subgrade provided that the water table is deep enough. Most often in Madagascar, the pervious subgrade is cohesionless, uniformly graded sand (coastal sands, Isalo, or Karroo formation sands). This type of material does not give lateral support to the pavement and, to avoid edge raveling, concrete or stone block alignments are recommended. As the subbase and the subgrade are pervious, lateral subdrainage is not required and the stone blocks or concrete curbs may be laid deep along the pavement edges.

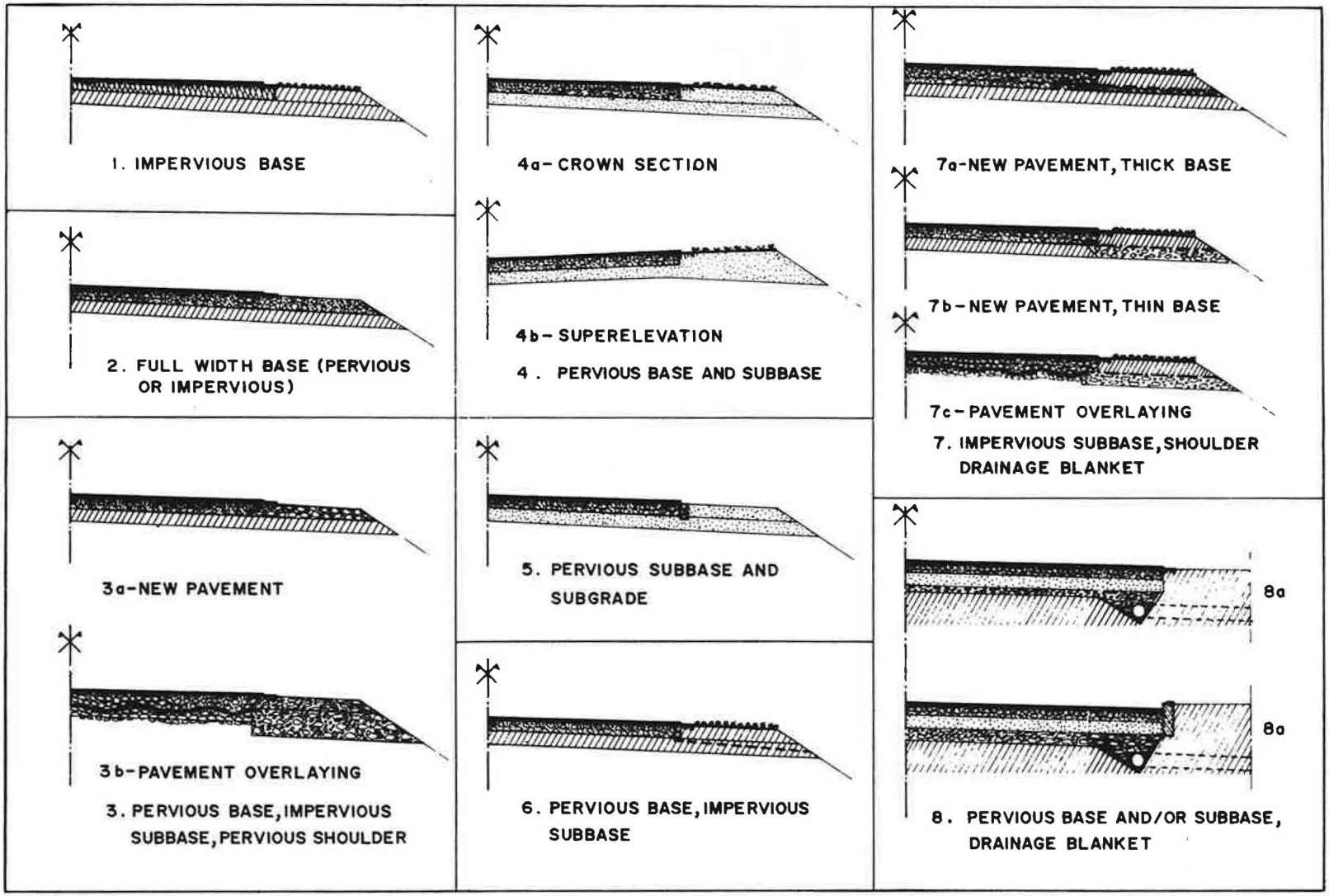

FIGURE 4 Catalog of typical pavement cross sections, Madagascar, 1986. Impervious pavement materials are represented by hatching. Pervious subbase (and shoulder) materials are most often sands and represented by dotting. Coarse materials in bases, shoulders, and blanket layers as well as turf surfacing of shoulders are represented by suggestive graphics.

Cross Section 6: Impervious Subbase and Shoulders With Drains

The pervious base receives a full-width surfacing to avoid or reduce ingress of rain water. Shoulder drains are conveniently spaced to provide adequate subdrainage.

The distance between successive drains should be designed based on the expected volume of water to flow through the pavement. Because frost is not a problem in Madagascar, the shoulder drains should simply be designed for an outflow rate at least equal to the design infiltration rate. Cedergren (*3,4*) related the design inflow rate to the 1-year frequency, hourly precipitation, whereas Ridgeway (*6*) considered infiltration through cracks. In this case, storm duration is more significant than intensity.

More research is required to establish reliable design infiltration rates in Madagascar. Tropical rainfall has both high intensity and long duration and the current bituminous seals are rather designed for minimum construction costs in the Red Island. The design infiltration rate is therefore most likely to take high values and the resulting drain spacing is most likely to be short.

As discussed previously, the shoulder drains should be carefully constructed for efficient operation.

Cross Section 7: Impervious Subbase, Shoulders With Drainage Blanket

This cross section has continuous, lateral drainage through a pervious layer under the shoulder.

Construction methods would be different depending on the thickness of the base. If the base is thick, it will have to be constructed in two layers. Construction of the upper shoulder layer may cause contamination and clogging of the inner edge of the drainage blanket at the base-shoulder interface. To avoid contamination during construction as well as later clogging during the service life, an intermediate geotextile membrane may be used.

The performance of turfing above the drainage blanket requires monitoring. Turfing and the shoulder upper layer may dry quickly during the dry season and lose bearing capacity and be damaged following saturation during the wet season.

Cross Section 8: Draining Blanket With Collector and Outlet Pipes

In some circumstances, particularly in towns, the pavement has to be placed in a previously prepared excavation. This type of trench pavement must have a special, properly designed subdrainage system with a drainage blanket as well as collector and outlet pipes.

Draining blankets are also required in cut sections where water inflow from the water table into the pavement could take place.

COMPARATIVE COSTS OF PAVEMENT DRAINAGE

In order to compare construction costs of pavement drainage alternatives, 1-km-long pavement stretches of the eight different standard cross sections described herein were considered. The assumed widths were 6.0 m for the pavement and 1.5 m for each shoulder. The wearing course was assumed to be a double surface dressing and thicknesses of 15 and 20 cm were taken for the base and subbase, respectively.

Unit costs were derived from 10 different contracts by averaging and updating to January 1986. However, the contracts did not have prices for fabrics and pipes and these had to be estimated.

After computation of construction costs for 1-km-long pavement stretches, a reference cross section was considered to derive comparative, relative costs expressed as percentages. As impervious stabilized bases would have high construction costs compared to crushed stone, the reference cross section was defined as Cross Section 1 with a 6-m-wide, impervious water-bound macadam. Thus, its construction cost was the lowest.

The base of standard Cross Section 6 was assumed to be a pervious water-bound macadam with the same unit price as the impervious water-bound macadam of Cross Section 1, so that only shoulder drains accounted for differences in construction costs of Sections 1 and 6.

A few different alternatives were considered within single standard cross sections, namely surfaced and unsurfaced shoulders for Cross Section 2, different spacings of shoulder drains with and without geotextile encapsulation for Cross Section 7 and one or two collector pipes for Cross Section 8.

Earth works as well as excavation costs for standard Cross Section 8 were not considered.

The relative construction costs of the different pavement cross sections are presented in Table 1. Because Cross Section 1 does not require any subdrainage, the cost percentage over 100 of the other sections can be considered as a good approximation of the cost of pavement subdrainage.

Table 1 indicates that the construction cost of pavement subdrainage is not excessive and is even low in some cases.

Extending the base course in full width (Cross Section 2) results in pavement costs that are about 25 percent higher than the reference cross section, but using low-cost pervious materials such as crusher run or natural gravels (Cross Section 3) only increases construction costs by 15 percent. Proper surfacing of shoulders costs 8 percent more.

Where pervious subbase natural materials are available, there is no extra cost for proper pavement subdrainage (Cross Section 4). The extra cost of Cross Section 4 only results from the higher cost of graded crushed stone over water-bound macadam in Cross Section 1.

The extra cost of Cross Section 5 results from additional costs of protection against edge raveling, which are relatively high and amount to 10 percent.

Pavement subdrainage by means of shoulder drains (Cross Section 6) has relatively low construction costs that vary with the drain spacing. Encapsulation of shoulder drains with geotextile is cost-effective and only represents an extra cost of 3 percent for the drain spacing of 5 m. For spacings of shoulder drains smaller than 5 m, the cost of this type of discontinuous lateral drainage tends to be higher than the cost of continuous lateral drainage through pervious crusher run or natural gravel shoulders.

The cross section with a continuous drainage blanket under the shoulders does not appear to be cost-competitive.

TABLE 1 RELATIVE CONSTRUCTION COSTS OF DIFFERENT PAVEMENT CROSS SECTIONS

Standard cross-section No.	Base course		Shoulders	Subdrainage		Construction cost (%)
	Width(m)	Materials				
1	6.0	impervious water bound macadam	impervious soil and turfing	not required		100
	6.4					102
2	9.0 extended over shoulders	graded crushed stone	unsurfaced	lateral, through pervious shoulders		124
			single surface dressing and sand seal			132
3a	6.0		pervious crusher run or natural gravel			115
4a			pervious sand: the same material as subbase	lateral, through pervious shoulders and subbase		103
5			blocks along pavement edges	vertical, through pervious subbase and subgrade		116
6		pervious water bound macadam	impervious soil and turfing	lateral, discontinuous, through shoulder drains	spacing=10m	105
					spacing= 5m	110
					spacing= 3m	117
				lateral, discontinuous, through geotextile encapsulated shoulder drains	spacing= 5m	113
7b		graded crushed stone	upper layer impervious soil and turfing lower layer pervious drainage blanket	lateral through drainage blanket		126
			as above with intermidiate geotextile			135
8a			impervious soil	drainage blanket, collector pipes on both sides and outlet pipes		157
				drainage blanket, collector pipe on one side and outlet pipes		146

The construction cost of a subdrainage system comprising a drainage blanket, collector, and outlet pipes appears to be the highest and not competitive for relatively thin, low-volume road pavements. However, this cross section is recommended for sections of trench pavement, and drainage blankets must also be considered for particular hill sections showing shallow, seasonal water tables or seepage from rock fractures.

CONCLUSIONS

Neglecting pavement subdrainage results in shorter pavement lives and experience indicates that under high tropical rainfall properly structurally designed, poorly drained pavements are subject to premature failure.

Poor subdrainage often results either from inadequate cross-section design or from the concern of the project engineer with minimizing construction costs.

On the basis of unit costs of 10 road construction and rehabilitation projects in Madagascar, a comparison has been made of construction costs of eight different cross sections and four methods of draining pavement systems.

The comparison showed that the low cost of percolation drainage through pervious subgrades can be canceled by the cost of protection against edge raveling in the case of cohesionless, uniformly graded sands.

Lateral subdrainage has a very low cost where natural pervious subbase and shoulder materials are available. Where shoulder materials have to be crushed or hauled for long distances, the cost of subdrainage may increase up to 15 percent. Extending the base course over the full width of the formation has a relatively high cost but is advantageous from the viewpoints of construction and performance.

Shoulder drains if properly designed and constructed have costs that are comparable to those of pervious shoulders but require manual work and careful construction.

Lower costs can be achieved by constructing pervious shoulders only in pavement sections and at the pavement sides where they will actually drain the pavement. The higher superelevated shoulders of curves should preferably be impervious.

As construction costs are sensitive to site location as well as economic environment, these conclusions resulting from cost comparison in Madagascar by the year of 1986 can prove wrong and inaccurate elsewhere and should not encourage design engineers to fail to evaluate design alternatives on a site-specific basis.

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Deflections of Lateritic Gravel-Based and Stone-Based Pavements of a Low-Volume Tea Road in Kenya

F. J. GICHAGA

Results are described from a testing program of a tea road on the slopes of the Aberdares where tea is the main cash crop. The 5.5-m-wide low-volume road has been carrying the equivalent of about 7,000 standard axles per year. Two experimental sections were identified on the road, one of which contained a lateritic gravel base and the other a stone base. The experimental sections were subjected to a detailed pavement condition survey to identify types and extent of distress features such as potholes, crazing, and edge failure after more than 10 years of service. In situ field tests were carried out on the subgrade, subbase, and base materials to determine in situ field density and moisture. The dynamic cone penetrometer and Clegg impact hammer were used to establish the in situ California Bearing ratio (CBR) values. In addition, pavement deflections and radius of deflected profile were measured using the Benkleman deflection beam. The results of the studies indicate that the experimental sections are still performing well after about 16 years of service. The deflection values obtained were below 100×10^{-2} mm, and the radius of curvature of the deflected profile was greater than 150 m. The pavement condition surveys tended to show that stone-based sections (standard construction) were marginally more distressed than the lateritic gravel-based sections (substandard construction). The study indicates that low-volume roads can be constructed using substandard materials such as lateritic gravels, for which the cost of the road is about two-thirds that of conventional road materials that meet specifications.

Kenya is heavily dependent on agricultural products such as coffee and tea, which are the main foreign exchange earners. The foreign exchange earned through exportation of these agricultural products is used to purchase imported technology in terms of equipment and expertise for faster economic growth of the country. Tea, which is a major cash crop, is grown in the highland areas where climatic conditions are favorable. Results are described of a testing program for a tea road on the slopes of the Aberdare mountains where tea is the main cash crop. Tea roads are constructed to provide all-weather access between the tea factories and the tea collection centers. These tea roads often have poor geometrics, sometimes with grades of 12 percent and more because of the need to provide them cheaply in difficult mountainous terrain. The tea roads are low-volume roads that provide the transportation system necessary for providing farm input service and for allowing agricultural products to reach the markets.

The road for which test results are reported is Gatura-Mataara Road, which is 5.5 m wide at an altitude of about 2040 m located in the Central Province of Kenya. The mean annual rainfall is about 1980 mm; mean monthly maximum temperatures range from 14°C to 21°C and mean monthly minimum temperatures range from 8°C to 10°C. The road is generally on a ridge for which alignment soils are dark red friable clays that are residual soils derived from recent lava and are relatively free draining. The water table is generally greater than 6 m below the ground surface. The objective of the testing program was to find out why some of the tea roads constructed with plastic laterites performed satisfactorily and to compare the performance of stone-based and lateritic-based pavements.

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EXPERIMENTAL SECTIONS

In general, the stone base was used in the Gatura-Mataara road in steeper grades in heavy-cut sections, whereas lateritic gravel base was used on the ridges with flatter grades or in light side hill cut sections. Topography appeared to make little difference in the performance of the two experimental sections except on sharp horizontal curves and on steep grades.

Gatura I Experimental Site ES9

This test section was selected on the Gatura-Mataara Road near Gatura shopping center to the west of a river bridge about 80 km north of Nairobi. The section, which is a 5.5-m-wide carriageway with 1-m-wide shoulders, was completed in 1974 and the pavement is made up of double-surface dressing on 130-mm-thick crushed-rock base on 100-mm-thick gravel subbase that rests on red soil subgrade. The subgrade generally exhibited a soaked CBR value of greater than 9 percent when compacted to 100 percent using a 2.5-kg rammer (light compaction). Most of the crushed stone for base construction was obtained from rock quarries of basalt in the Aberdares. The crushed stone was laid by a paver and compacted by a vibrating roller and a 14-ton three-point flat roller. The surface dressing was carried out using 1,000-gal bitumen distributor, self-propelled chip spreader, pneumatic-tired roller, and tandem roller. The road was carrying about 7,000 equivalent standard axles per year in 1983 and, at a growth rate of about 5 percent per year by 1990, the pavement had received about 0.11×10^6 cumulative standard axles since construction. A standard axle in this case represents 8165 kg (18,000 lb).

Gatura II Experimental Site ES10

This test section, which is also a 5.5-m-wide carriageway with 1-m shoulders, was selected on the same road as Gatura I about 5 km beyond Gatura I (ES9). The section is made up of double surface dressing on 150-mm-thick lateritic gravel base on red-soil subgrade with CBR values of about 5 to 20 percent. The lateritic gravel had a CBR value of 43 percent and was won by ripping and dozing into stockpiles. The gravel was either a weathered tuff or basalt that occurred sporadically in the vicinity of the roads or laterite formed in residual volcanic soils. Front loaders were used to load the gravel into tippers, and spreading was carried out by graders while watering was done by self-propelled bowsers. The lateritic gravel was exposed to 6 months of traffic and weather (i.e., one rainy season) before surface dressing. The gravel base surface was prepared by mechanical brooming and wetting before priming and surfacing. The road section has been subjected to the same traffic loading as Gatura I, i.e., 0.11×10^6 cumulative standard axles, from 1974 to 1990.

The *road design manual* in Kenya (*1,2*) specifies that gravel for use as base should have a minimum CBR value of 50 percent (4 days' soak), whereas the lateritic gravel used in the Gatura II road section had a CBR value of 43 percent. The *Road Design Manual* again requires that the plasticity index (PI) of base gravel for wet areas be a maximum of 15 percent, whereas the lateritic gravel used for the Gatura II road section had a PI of 18 percent. Thus, according to the *Road Design Manual* (*2*), the lateritic gravel used for Gatura II Section is substandard.

With a CBR value of 9 percent, the required stone-based pavement structure is 125 mm of crushed stone base on a 100-mm subbase, according to the *Road Design Manual* (*2*), and thus the design of the section (Gatura I) satisfies the specifications from the *Road Design Manual*. Road Note 31 (*3*) specifies 150 mm of road base on 100 mm of subbase, in which case the base layer is slightly thinner than specified in Road Note 31. Table 1 presents material characteristics for the two experimental road sections (*5*).

EXPERIMENTATION

The following field tests were carried out on the two selected test sections described earlier.

Benkleman Deflection Measurements

Benkleman deflection beams were used to measure elastic deflection of the pavement surface using rebound deflection procedure. In the case of road pavements, deflection tests were carried out on 10 marked test points on a 60-m road section. Test points were marked on the road surface at five cross sections spaced at 15-m intervals. The test truck (a 7-ton tipper) was loaded to give a rear axle load of 6350 kg (14,000 lb), and rear tire pressures were set at 586 kN/m^2 (85 lb/in.^{2}); the tire size used was 900 × 20D. The standard axle is again taken as 8165 kg (18,000 lb).

Measurements of Radius of Curvature

The radius of curvature of the deflected pavement profile was obtained during the Benkleman deflection measurements. The

TABLE 1 MATERIAL CHARACTERISTICS FOR EXPERIMENTAL ROAD SECTIONS (*5*)

Characteristic	Subgrade	Lateritic Base	Stone Base	Surface Dressing
Optimum moisture content (%)	36–63	16–25		
In situ moisture content (%)	32–52	7–14		
Max. dry density (kg/m^3)	1030–1230	1750–1900		
Relative compaction (%)	100–107	99–105		
Liquid limit (%)	54–82	55		
Plasticity index (%)	13–24	18		
CBR (%)	5–20	43		
Swell (after 4 days' soak) (%)	0.2–0.3	–		
Grading analysis (% passing)				
38mm		100		
19mm		91		
9.5mm		80		
6.4mm		63		
(No. 7) 2.4mm	100	47		
(No. 36) 0.425mm	98	24		
(No. 100) 0.15mm	82			
(No. 200) 0.075mm	76	16		
Surface Dressing				
Prime coat				RCO
First seal				MC5
Chippings				70 m^2/m^3 (20mm)
Second seal				MC5
Chippings				85 m^2/m^3
Aggregate crushing value (%)			32[a]	20
Flakiness index (%)			30[a]	28
Bitumen affinity				Good

[a]Minimum requirements.

load test truck was driven at creep speed and stopped at premarked intervals to enable measurement of rebound deflection using the influence line technique and thereby obtaining the longitudinal deflection profile. The longitudinal profile data were used to compute the radius of curvature and also the equivalent modulus of the pavement structure.

Measurement of Field Moisture and Density

Field moisture and density were also measured. Trenches were excavated across the pavement to enable carrying out in-situ tests. The trenches extended from the road centerline to the verge on either side of the road. The in-situ tests were carried out for the various layers of the low-volume road pavement structure—namely, subgrade, subbase and base layers. The field sand replacement method and the nuclear density meter were used to measure the in-situ density and moisture contents.

Measurement of Field CBR Values

Field CBR values were determined using the dynamic cone penetrometer (DCP) and the Clegg impact hammer (CIH). Trenches were cut across the road pavement as described earlier and the DCP and CIH were used to obtain sounding values that were converted into CBR values. The DCP and CIH soundings had been calibrated against CBR values to enable conversion of DCP and CIH values into CBR values. The DCP and CIH soundings were made in each layer of the pavement structure—namely, base, subbase, and subgrade.

Pavement Condition Survey

A detailed pavement condition survey for the two experimental sections was made in 1989; the objective was to identify potholes, edge failure, crazing, and other forms of pavement distress. The road had received little or no maintenance since completion.

DISCUSSION OF RESULTS

The deflection measurements carried out on the two test sections gave the deflection values presented in Table 2. The corresponding radii of curvature are also presented in Table 2. In order to characterize the bearing capacity of the pavement structure, the parameter equivalent modulus (Eq) was computed for the two test sections for which results are also included in Table 2. Equivalent modulus, which is based on Burmister's two-layer theory, is defined as follows (*4*):

$$\mathrm{Eq} = 10^{r-1}\mathrm{El}(R/D)^{r/2} \tag{1}$$

where

Eq = equivalent modulus of a pavement structure (kg/cm^2),
El = elastic modulus of the upper pavement layer (kg/cm^2),
R = radius of curvature (m),
D = rebound deflection ($\times 10^{-2}$ mm), and

$$r = 1/[1 + \log(\mathrm{El}/1018)] \tag{2}$$

Wambura (*4*) has also shown that

$$(\mathrm{El})^B = (R/0.056)(D/5{,}800)^A \tag{3}$$

where

$A = (1 - X)/(1 - Y)$,
$X = 0.86 \log h - 0.474$,
$Y = 0.493 \log h - 0.41$,
$B = (1 - A)$, and
h = thickness of the upper pavement layer (mm).

Given the value of h, one can compute X and Y and hence obtain A and B. With A and B and the values of R and D from field measurements, El can be computed and hence r can also be computed. Thus, the value of Eq can be obtained from Equation 1.

Results presented in Table 2 indicate that the pavement structure made with a substandard lateritic gravel base (Gatura II) is marginally weaker than the pavement structure made of crushed stone base (Gatura I) on the basis of deflection and the equivalent modulus.

The results of the pavement condition survey indicated that potholes in the lateritic gravel-based section (Gatura II) were usually shallow and roughly circular. The potholes in the stone-based section (Gatura I) were deeper, much larger, and less regular in shape. The road was not subjected to heavy traffic and little maintenance had been done since completion of the road. In some cases, the verge adjacent to the road had been eroded away, giving a height difference of up to 300 mm between the road pavement level and the verge. On the basis of the pavement condition survey, it appeared that the lateritic gravel-based section (Gatura II) resulted in a stronger pavement than the crushed stone-based section (Gatura I).

TABLE 2 TEST RESULTS OF DEFLECTION AND RADIUS OF CURVATURE MEASUREMENTS FOR GATURA I AND GATURA II

Test Section	Pavement Thickness (mm)	Mean Rebound Deflection (D) $\times 10^{-2}$ (mm)	Radius of Curvature (R) (m)	Equivalent Modulus (Eq) (kg/cm^2)
Gatura I (ES9) (stone base)	130	49	166	2137
Gatura II (ES10) (lateritic gravel base)	150	61	200	1836

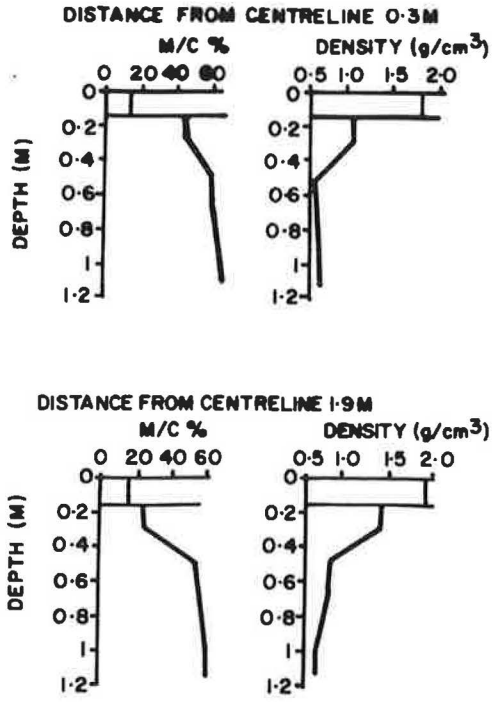

FIGURE 1 Moisture and density variation with depth for ES10 (Gatura II).

An interesting feature of the lateritic gravels was that they were plastic, becoming slippery because of rainfall, but on drying the fines were blown away leaving a densely packed stable gravel in a matrix of fines. This densely packed stable gravel base gave the pavement a strong structure that was difficult for surface water to penetrate. This is believed to be the main reason for the good performance by the lateritic gravel-based pavement structure.

Moisture and density measurements for Gatura II (ES10) gave results shown in Figure 1. The results indicate that moisture is higher in the subgrade than in the pavement, which is as expected.

In situ CBR values at the various depths in the layers of the pavement structure for Gatura II (ES10) with lateritic base were estimated using the DCP and CIH, giving the results shown in Figure 2, from which the CBR value obtained was close to the design CBR using the Road Note 31 (*3*) procedure. The results of the study also indicated that the CBR value of the subgrade immediately beneath the lateritic base in the case of Gatura II (ES10) had increased from values of about 5 to 20 percent to values of 30 to 50 percent. This phenomenon would require further detailed study to establish the cause of substantial increases in CBR value of subgrade.

According to Henry Grace and Partners (*5*), the cost per kilometer of various elements of construction for the two test sections was as follows (1972 prices):

- Cost of preliminaries, earthworks, culverts (excluding bridges), plus surface dressing = Kshs. 169,473 (common for both alternatives);
- Additional cost for crushed stone base alternative = Kshs. 180,744; and
- Additional cost for lateritic gravel base alternative = Kshs. 55,677.

Thus the total costs per kilometer were as follows:

- For crushed stone base construction—Kshs. 350,217
- For lateritic gravel base construction—Kshs. 225,150

The cost of lateritic gravel base construction (substandard construction) that performed reasonably well was about two-thirds that of the crushed stone base construction (standard construction).

CONCLUSIONS

The results of the study on lateritic gravel-based and stone-based pavements of the low-volume tea road led to the following conclusions:

- Rebound deflection values for both test sections were below 100 × 10^{-2} mm;

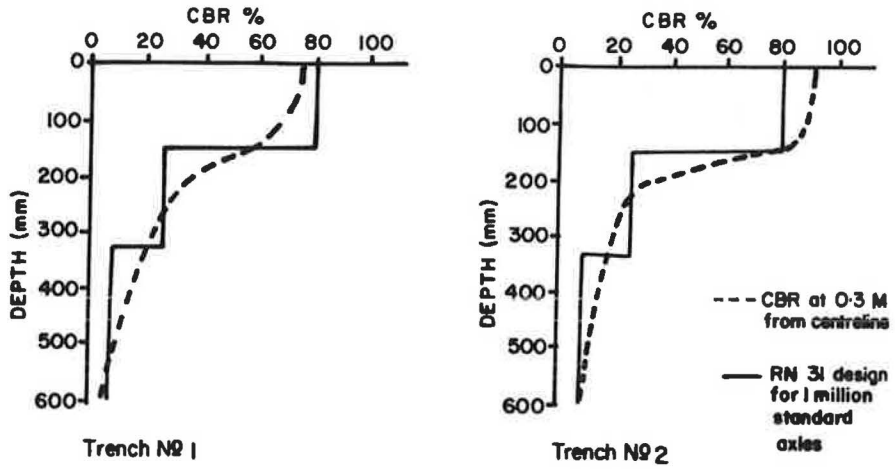

FIGURE 2 Variation of CBR with depth below surface for ES10.

- The radii of curvature of the deflected pavement profiles were above 150 m for both test sections;
- The experimental sections were still performing satisfactorily after 16 years of service during which they had carried about 0.11×10^6 standard axles; and
- Low-volume roads can be constructed using substandard materials such as lateritic gravel; such construction would give reasonable service and would cost must less than the standard pavement structure.

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Low-Cost Asphalt-Surfaced Roads

E. JAMES MANDIGO, STEPHAN JOHNSON, AND GORDON KELLER

A variety of low-cost asphalt road-surfacing techniques and a unique technology transfer method used to gather information are discussed. The premise is that the employees of the U.S. Department of Agriculture, Forest Service, have a large amount of knowledge and experience on low-cost asphalt-surfaced roads; however, little of it is formally documented. The technology transfer was accomplished by creating an electronic forum that is readily available to this large group of employees. The process facilitated an open discussion of the topic and brought together knowledge and experience from this diverse group of engineers, who are located throughout the United States. The data include numerous processes such as asphalt surface treatments, oil mats, road mixes, desert mixes, and sand seals. Design methods as well as test data are presented. The information, while largely originating from experiences and examples west of the Rocky Mountains, has application to many other locations.

Asphalt concrete has become the standard road-surfacing material for primary, secondary, and many local roads in the United States. Many miles of roads surfaced with native material and aggregates throughout the nation still need to be upgraded.

Asphalt concrete (AC) surfacing is desirable but costly when compared to aggregate surfacing. Native materials and aggregate surfacing are less desirable but cheaper to construct. There are many miles of existing treated roads throughout the United States that fall between these two categories. Many are surfaced with a mixture of asphalt material combined with a small amount of graded aggregate or native material. Often they have performed well for many years. These surfaces are referred to as "low-cost asphalt-surfaced roads" (LCASRs).

Processes included in LCASRs are numerous and are referred to by a variety of names. Some of them are well known and many are local. The processes include asphalt surface treatments, oil mats, road mixes, desert mixes, and sand seals.

Quite a bit of information is available from the Asphalt Institute and other sources for the relatively higher-cost surfaces, but little information seems to exist for the less costly alternatives. These lower-cost "alternatives" are found, in some cases, to perform as well as the more costly processes. Local roads surfaced with sand and asphalt and decomposed granite-asphalt mixes have been observed on the Inyo, Sequoia, and Mendocino National Forests in California (unpublished internal reports, U.S. Forest Service). The problem is to find out which will be successful for a particular situation, under what conditions, what specifications should be used, what are the costs, and what materials will work.

Another class of low-cost surfaces consists of chemical treatments with materials such as lignin sulfonate, magnesium chloride, and proprietary products such as Biocat. It was originally intended to include this class of materials herein; however, insufficient data were gathered during this study to adequately cover the subject.

The Forest Service has a tremendous amount of experience and knowledge in low-volume road technology; however, much of it is not formally documented. As much of this information as possible was pulled together along with references and other data from local road agencies and individuals working in these areas.

TECHNOLOGY TRANSFER PROCESS

To gather information, a unique technology transfer (T2) process was begun in the Pacific southwest region (California) of the Forest Service in May 1988. The process consists of engineering T2 coordinators located agencywide who are in contact with road specialists in their local areas and are linked together by an interactive computer network. Information received from them was then shared through the same network in summary form and filed for compilation. The information received included

- Specific LCASR project experience;
- References, texts, literature, newsletters, etc.;
- Names of individuals who have LCASR experience; and
- LCASR design guides.

A large amount of comments and information was received, including design guides for asphalt surface treatments and detailed plans and specifications for rehabilitation work on distressed pavements. In addition, a number of references were suggested for use on various parts of LCASRs.

Most of the information came from 32 Forest Service employees located on 18 national forests in six regions plus the Washington office. The employees consisted of forest engineers, geotechnical engineers, materials engineers, maintenance engineers, and others.

The T2 process had a dual purpose: (a) to gather information, and (b) to provide immediate benefit through feedback of information and contacts for help to those in the agency who were designing, building, and maintaining LCASRs.

The information gathered through this effort and through additional literature searches has been compiled into a synthesis of options for LCASR design, costing, construction, and maintenance.

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TYPES OF LOW-COST ASPHALT-SURFACED ROADS

Road Mixes

The term "road mixes" means different things to different people; therefore a definition is needed. Road mixes are considered to consist of treatment of the top (1 in. to several inches) of the existing road surface by means of adding asphalt or other additive, mixing, and compacting in place. In some cases, additional surfacing material may be added before treatment; however, the distinction between road mixes and other types of road surfaces is that the mixing is done on the road, as the name implies.

Road mixing is normally done in one of two ways: (a) by a traveling pugmill or cross-shaft mixer, or (b) by windrow mixing with a motor grader (blade). The traveling mixer is preferred because the end product is a more uniform mix and will have better depth control. Unfortunately, traveling mixers are hard to find at times. A good grader operator can produce an acceptable mix, but good grader operators with blade mixing experience are also scarce [Gordon Keller, Mailing 2 in *Low Cost Road Surfaces* (LCRS) *Data Summary*, unpublished; all LCRS mailings referred to in this paper are available from E. J. Mandingo, U.S. Forest Service, Pleasant Hill, California].

A road-mixing alternative will normally not be competitive with other alternatives if much of the crushed aggregate must be imported any distance. If the road requires new material, consider mixing the aggregate with asphalt or other additive at the aggregate source by means of a stationary pugmill. The end product will be better, although costlier than if existing material can be treated in place.

One of the common pavements found in the Sierras is what is sometimes called a "desert mix." It is quite common to find a fairly major rural road of this type that has been handling traffic for 15 to 20 years with little distress. Often the pavement consists of a mixture of asphalt and native decomposed granite (DG) and has been chip sealed periodically similar to higher-type pavements. History of the construction of these roads is hard to find and the practice does not seem to be common anymore. One of the major reasons for the success of these roads is the structurally sound nature of the DG soils because of their nonplastic and generally well-drained characteristics. The asphalt was probably a cut-back (MC or SC). The MCs may be a problem now because of air quality restrictions.

A similar road surface was produced with a cinder aggregate road mix constructed on the Rice Canyon Road on the Tahoe National Forest in about 1973 (Stephan Johnson, Forest Service, unpublished data). An SSK-h cationic emulsified asphalt (current designation is CSS-1h) was mixed on the road with a Woods™ traveling pugmill. An asphalt content of approximately 14 percent (by total weight) was required. The high percentage was caused by the porous nature of the cinders and their low weight (110 lb/ft^3). The mixture appeared initially to be wet from the high percentage of fluids, but apparently worked well, because it is still in use today.

Other native materials have been used for road mixing with various degrees of success. Volcanic cinders and pumice have also been used in eastern Oregon by counties and by the Forest Service (John Lund, Oregon Institute of Technology, unpublished data).

Cold Mixes

Cold-mix asphalt pavements are on the high end of LCASRs and were quite popular in the mid-1970s. Many miles were constructed by the Forest Service in both the Pacific Northwest and the Pacific Southwest Regions (in Oregon, Washington, and California). Cold mixes are generally produced at a stationary pugmill mixer often set up at the crusher or stockpile site. The mix usually consists of a crushed-rock aggregate of a single gradation blended together with an emulsified asphalt that is metered into the aggregate in the pugmill.

A cold-mix project was constructed on the Tom Young road (Ron Andrus, LCRS Mailing 4, unpublished) many years ago, which used SSK-h emulsified asphalt mixed with screened cinders. The pugmill was set up downstream from the screen and the mix was placed on the road with a conventional paving machine to 3-in. depth. The material is still in place; however, most of the original cold mix has been recycled into base material and more base was added to correct the deficient structural strength. Approximately 3 mi of the original surface remains in use today.

Another interesting construction material that is planned for use in Routt County, Colorado, is a tar sand (a naturally occurring petroleum-sand mixture) imported from beds near Vernal, Utah. Routt County plans to crush the material when cold, blend 40/60 with crushed aggregate, then lay it on the roadway and compact it. This technique has been used around Vernal, Utah, and Dinosaur, Colorado, with excellent results and appears to cost less than half of what it would for conventional asphalt concrete. Transportation costs and limited sources will limit its use (Rex V. Blackwell, LCRS Mailing 6, unpublished).

Because control of most of the cold-mix processes is less precise than hot-mix asphalt concrete, the strength factors used to compute thickness are generally considered to be between 70 and 80 percent of those used for hot mix. This procedure results in a thicker pavement section, reducing the cost advantage of the cold mix. In situations where existing structural strength of the road is adequate, cold mixes usually have an economic advantage over hot mixes. Other solutions such as surface treatments can prove to be more economical in some moderate traffic and environmental conditions.

Technical information for the design and construction of cold mixes is published by the Asphalt Institute. Two publications that are especially useful are *Asphalt Cold Mix Manual* (*1*) and *Basic Asphalt Emulsion Manual* (*2*).

Penetration Treatments

Penetration treatments or prime coats are normally not considered as a surfacing type by themselves, but are included here because of some limited use in isolated situations. Occasionally, a prime coat is applied preparatory to paving or a surface treatment and it becomes necessary to carry traffic for an extended period of time on its surface. This practice, although not recommended, can be successful providing the

base is adequate; cross section, vertical alignment, and horizontal alignment are not too severe; and traffic weights and speeds are not excessive. Ambient temperature as well as the tightness (i.e., the ability of the prime to penetrate into the base surface) also play an important part in the success. If ambient temperatures are high, some cover or blotter material will be necessary to prevent bleeding and pick-up of the asphalt.

Asphalt used for prime coats is usually an MC cutback; however, because of environmental considerations discussed later, emulsions are becoming more widely used for this purpose. A good asphalt reference such as *The Asphalt Handbook* (*3*), published by the Asphalt Institute, should be consulted for design of prime coats.

Asphalt Surface Treatments

Asphalt surface treatments are probably the most widely used of the LCASR treatments on Forest Service roads in the west during the last decade or so. There are many names, many of which are descriptive of the individual process. Some of the more common ones will be described here.

Fog Seals

Fog seals are generally a light application of emulsified asphalt applied to the surface of an AC pavement. They were used extensively in the west during the 1950s and into the 1960s to seal new pavements immediately after paving. They fell out of favor with many agencies, however, because of the tendency for the pavement surfaces to become very slippery when wet and to bleed easily in hot weather. These problems were largely caused by excessive application or inappropriate choice of seal materials because of inadequate or unused surface evaluation methods. The cure was to burn the excess asphalt off with a large vehicle-mounted propane burner, which also caused additional problems of high cost and air pollution. Fog seals have a limited place in the rehabilitation of oxidized or otherwise distressed pavements as well as for low-traffic areas such as parking lots.

Seal Coats

Seal coat is a generic term that is commonly used for a number of surface treatments applied to a paved surface and includes fog seals and chip seals.

Chip Seals

Chip seals (or single-chip seals) commonly refer to sealing of existing pavements, similar to fog seals, except that additional emulsified asphalt is used and it is blotted or covered with a fine aggregate; usually a single-size crushed chip with a maximum size of ¼ or ⅜ in.; ½- and ⅝-in. chips have been used successfully on roads with heavy truck traffic to extend chip life by reducing embedment.

Successful chip seals have also been constructed using natural occurring sands and decomposed granite on national forests in the southern and central Sierras as well as with cinders and pumice in northern California and central Oregon. Dust is often a problem with native materials, however. Dust-free aggregate is important to chip seal success.

The first seal is often delayed until 5 to 7 years after the road is paved. Subsequent seals are usually needed on about a 5-year cycle. Chip seals avoid both the common drawbacks of the fog seals because of their superior traction characteristics and less chances for an overly rich application.

Perhaps this discussion on pavement sealing seems out of place for LCASRs; however, sealing is one of the primary repairs for badly deteriorated pavements and as such may be useful to a road agency to stabilize and retain some of the value of an apparently lost pavement.

Single-chip seals can also constitute the entire LCASR treatment applied directly to an aggregate base or a native material road surface. In this form, they are applied like the penetration treatments except that the quantity and grades of asphalt are selected to retain the cover aggregate as well as penetrate the base. Another similarity is the need for a well-drained base that possesses structural strength adequate for the loads that will be imposed on it (Forest Service LCRS Mailings 4, 6, 8, 12, 15, and 18, unpublished). A single-chip seal has little structural strength; however, it is flexible and can survive some base deflection without failing. Chip seals used in this manner can also be constructed with a variety of aggregates, both manufactured or native, and can be expected to last for many years of light service if properly maintained. Maintenance needs vary depending on many variables including

- Traffic volume, weight, and speed;
- Weather; and
- Road alignment, vertical and horizontal.

Forest Service engineers in the Northern Region (Montana, northern Idaho, North Dakota, and northwestern South Dakota) (Steve Monlux and Bob Hinshaw, LCRS Mailing 8, unpublished) recommend the first maintenance seal coat be applied in 1 to 3 years with additional treatments every 3 to 5 years. Actual application of seal coats, particularly in the west coast states, is usually less frequent.

Multiple-Chip Seals

Multiple-chip seals are the most common and probably the most successful of the LCASR types that were reported. The reasons are several: they are more common (therefore contractors and engineers are more experienced in their design and construction), the construction process is somewhat less critical than for cold mixes, and reliable procedures are available and are well documented. A multiple-chip seal may consist of from two to as many as five applications of asphalt with alternating spreads of aggregate. Typically two or three applications are used with the first one being a penetration spread. Obviously, the more numerous applications are more costly and will not compete well from a cost standpoint when compared to hot- or cold-mix pavements.

Life expectancy of double-chip seals on a well-drained, structurally adequate base are reported to be 10 to 12 years,

even more than 20 years (Ron Andrus, LCRS Mailing 5, unpublished) provided traffic weight and speeds are not excessive, road alignment is gentle, and if routine seal coats are provided on a regular cycle as described for single-chip seals.

Traditional designs for multiple-chip seals usually call for clean-crushed aggregate screened to single sizes such as ¾ to ⅜ in., ½ in.—#4, and ⅜ in.—#6. The problem is that these gradations are usually quite expensive to manufacture and may not compete well economically with higher-type pavements. To be cost-effective, LCASR designs must take advantage of available local materials and be customized to fit the site conditions.

LIFE CYCLE COSTING

Life cycle costing is a useful technique and should be used to select the most feasible alternatives of construction for LCASRs. The life cycle costs should be based on preliminary designs and cost estimates together with predicted maintenance and operation costs (Milada, LCRS Mailing 4, unpublished). This method provides a convenient way to compare alternative designs including the various costs of constructing, operating, and maintaining them. Operation and maintenance costs are often as critical to a low-volume road agency as initial construction costs.

DESIGN CONSIDERATIONS

Too often, the common practice for selecting an asphalt surface treatment consists mainly of using design information from a previously constructed project. This practice may be expedient and in some cases successful; however, it is frequently the cause of failures or poor-quality surface treatments.

Proper design should include site- and project-specific information on

- Composition, condition, and structural strength of the base or underlying road material;
- Aggregates and asphalt to be used; and
- Environment of the project location.

The structural strength of the road before an asphalt treatment is applied is probably the most influential element in the success of the LCASR. The many methods available to evaluate existing strength are beyond the scope of this paper and it is assumed that a satisfactory method has been used prior to LCASR design.

For strength information, the Forest Service generally performs various field investigations such as deflection testing, sampling, and testing of base and subgrade materials. The field data obtained are then correlated to *Forest Service Handbook 7709.11*, Chapter 50—PAVEMENTS (*4*), which is a primary method used to design aggregate bases, asphalt pavements, and aggregate surfaces.

TYPICAL SURFACING DESIGNS

Following are three proven design methods for asphalt surface treatments that were supplied by contributors to the LCASR study.

Forest Service (California) Design Recommendations

The following procedure is a chip seal design method that can take a large percentage of the guesswork out of determining aggregate and emulsion spread rates (Stephan Johnson, LCRS Mailing 9, unpublished).

The variables in determining spread rates are as follows:

- Void space between the particles in the aggregate (chips),
- Gradation of the aggregate,
- Flatness of the aggregate (flakiness),
- Texture of the surface being chipped,
- Density of the aggregate (bulk specific gravity), and
- Amount of traffic.

There are other factors that have an influence on the success of the job. They include time of the year the job is constructed, and environmental factors such as temperature, humidity, and wind.

The recommended design procedure is contained in Asphalt Institute MS-19 (*2*) which was also published by FHWA (*5*). These publications are virtually identical, but the second edition of MS-19 does not contain the design formulas and accompanying text.

The sieve analysis requires a ¼-in. sieve and the flakiness index requires the use of a gauge or slotted sieves as described in the previously mentioned manual. Flakiness index gauges and sieves can be obtained from engineering and materials testing supply firms.

Plumas and Eldorado National Forest Typical Designs

The following recipes represent the typical practice from Plumas National Forest for constructing a conventional double-chip seal on an aggregate base, and a double-chip seal method developed on the Eldorado National Forest. These include the normal construction sequence, typical material type, quantities used, and other factors affecting cost and performance (Gordon Keller, LCRS Mailing 7, unpublished).

Plumas Double-Chip Seal

1. Subgrade or base (aggregate) should be structurally sound and well drained.
2. Compact base to 95 percent of compaction by the procedures of the AASHTO Test T–180 and obtain a clean, smooth surface.
3. Add penetration treatment—0.40 to 0.50 gal/yd² of MC-250 asphalt for coarse aggregate or 0.35 gal/yd² of MC-70 asphalt for fine aggregate or soils.
4. Let cure 5 to 10 days; blot lightly as needed to accommodate traffic.
5. Apply 0.25 to 0.35 gal/yd² of CRS-2 asphalt.
6. Immediately apply 20 to 30 lb/yd² of ½ in. × No. 4 chips. Uniformly graded chips are best. Also, coarser chips wear longer because they provide more depth, less embedment, and less bleeding, although they require more materials.
7. Roll with pneumatic tire roller.

8. Let cure for 24 (minimum) to 72 hr. The second asphalt and chip application can follow immediately after rolling the first course if strict, complete traffic control can be kept for 24 to 48 hr.
9. Broom off excess chips.
10. Apply 0.20 to 0.25 gal/yd² of CRS-2 asphalt (preferably with latex additive).
11. Immediately apply 15 to 25 lb/yd² of ⅜ in. × No. 6 chips.
12. Roll with pneumatic tire roller followed by steel wheel roller (optional for a smooth surface).
13. Broom off excess chips (optional on low-speed roads).
14. Control traffic speed to 5 to 15 mph for the first 24 hr (4-hr minimum).

Typical asphalt quantity is 0.95 gal/yd² (0.40 gal/yd² penetration application + 0.55 gal/yd²); typical aggregate quantity is 50 lb/yd²; and typical cost is $2.05/yd² (in 1989 dollars). The cost varies with the size and location of project.

With good grade, alignment, and structural conditions, 20 to more than 30 million board feet (mmbf) of timber haul can be expected over double-chip seals, with little damage. Normal conditions will require some patching and maintenance to accommodate this traffic.

Forest Service engineers frequently use millions of board feet to describe haul quantity on roads in areas where timber harvesting is the predominant commercial activity. Extensive harvest records have provided accurate timber volume per loaded log truck (usually estimated to be 5,500 board feet), which weighs about 14 kips per 1,000 board feet (mbf) including the weight of the truck. For structural design purposes, this value equates to about 650 equivalent 18-kip axle repetitions per mmbf. Detailed information and example computations are included in *Forest Service Handbook 7709.11*, Chapter 50 (*4*).

The second course of chips should be one-half the size of the first course.

Large chip seal projects should be designed to consider specific aggregate type, shape, weight, voids, absorption, residual asphalt content, etc.

Weather limitations are important, and warm, dry weather is critical for a successful project. Construction should be before September 15, particularly in northern climates.

Chip seal life is substantially shortened on grades over 12 to 15 percent or if there is tire chain use in winter.

Chip seals are most cost-effective and appropriate on roads with moderate-to-light sustained traffic.

Eldorado Style Chip Seal

1. Provide a ¾-in. aggregate leveling course and additional base aggregate as needed for structural support.
2. Steel wheel (vibratory) roll for compaction and smoothness.
3. Prime with MC–800 asphalt (usually 0.50 gal/yd²).
4. Chip/blot immediately with ⅜-in. chips at 18 to 25 lb/yd². Use ample amount of chips.
5. Roll with pneumatic (rubber tire) roller.
6. Let cure 7 to 10 days.
7. Broom off excess chips.
8. Apply 0.15 to 0.25 gal/yd² of CRS-1 or CRS-2 asphalt with latex additive.
9. Chip again with 18 to 25 lb/yd² of ⅜- or ¼-in. chips.
10. Rubber tire roll again.

Typical asphalt quantity is 0.70 gal/yd²; typical aggregate quantity is 45 lb/yd²; and typical cost is $1.75/yd² (in 1989 dollars).

The Eldorado National Forest has been using this treatment for several years and reports timber haul of 20 to 60 mmbf without damage to the surface. The high performance (over 30 mmbf hauled) may result in part from the treatment being applied to very stable old roads and prompt repair of broken areas. Estimates of timber haul in excess of 30 mmbf can only be expected under ideal conditions.

Less asphalt use and therefore lower cost than a typical double chip seal are advantageous. Chips can be placed immediately on the MC-800 asphalt without a typical penetration treatment which avoids possible contamination of this layer.

This recipe may be improved by using ½-in. chips for the first course and ⅜-in. chips for the second course.

CAUTIONS AND ADVICE FOR CHIP SEALING

There are numerous problems to be considered in the design and construction of a chip seal. Some of the possible problems associated with chip sealing over an aggregate surface follow (Stephan Johnson, unpublished data).

Assumptions

There are two main assumptions that are built into a chip seal design. The first is that the chips are one-sized. A one-sized aggregate as defined by McLeod (*6*), is an aggregate that has a gradation of 60 to 70 percent passing the specified sieve and retained on a sieve having an opening that is 0.7 of the specified size. The second assumption is that the surface receiving the chip seal is hard enough that the aggregate particles are not embedded into the existing surface.

Aggregate Base

This layer together with the subbase, if any, and the subgrade provide the entire structural strength in a typical project that includes a chip seal over aggregate base. No structural strength is considered as derived from the chip seal.

Things that can go wrong include the following:

Chips become embedded in aggregate base. The biggest problem occurs when the aggregate base allows the chip to be partially embedded in the surface of the aggregate and therefore the amount of asphalt material needed to hold the chip in place is reduced. The most obvious cause is that the aggregate base is not adequately compacted.

The aggregate base is out of specifications. The problem is the gradation is not within specifications and allows the chip to be either embedded in the surface or the base is of insufficient strength and shears from torque or centrifugal forces.

Degradation of aggregate base. The quality of the aggregate is poor and it degrades at an accelerated rate.

Inclement weather. If the chip-sealed surface is used during inclement weather and there is not sufficient strength in the aggregate base, the surface will be deformed and probably break up.

Prime

The purposes of the prime are many. The prime preserves the aggregate base in a compact state by maintaining the moisture content, which allows construction and limited public traffic use without causing appreciable damage. The prime penetrates the surface of the aggregate and adds to the total thickness of the asphalt-treated material. Also, the prime provides a surface to which the chip seal will readily adhere. The traditional prime is an MC-70 cut-back asphalt. The asphalt spread rate depends on the porosity or amount of fines at the surface of the aggregate. The moisture content of the aggregate should be optimum and the surface of the aggregate base should be damp just prior to the application of the prime coat.

Things that can go wrong include the following:

- If too much cut-back asphalt is applied and the entire amount does not penetrate, a puddle of residual asphalt remains on the surface of the aggregate base. The end result of these areas is that when the remainder of the chip seal is applied, those areas will be flush with excess asphalt and will probably bleed under traffic.
- If traffic is allowed to use a primed road too soon, the prime may adhere to the tires and cause extensive damage to the treated surface.
- In some areas local air pollution control regulations prevent the use of MC cut-back asphalt. Substitutes that have been used are emulsified asphalt and SC cut-back asphalt. Emulsified asphalt will not penetrate the aggregate surface.
- Another major concern is if insufficient time is allowed before placing the chip seal. If the MC diluent (kerosene) is not allowed to evaporate prior to placing the chip seal, the entire chip seal is softened by the remaining diluent. This is especially true when an SC cut-back asphalt is used because the diluent is usually diesel fuel. Environmental conditions greatly influence the time necessary for the prime to cure.
- Allowing too much time between the prime and the subsequent chip seals may allow existing traffic to cause extensive damage to the prime and, therefore, the surface of the compacted aggregate base.

Asphalt Emulsion/Aggregate (Chip) Seal(s)

Normally, a double-chip seal is applied over an aggregate base. Each subsequent layer uses aggregate one-half the nominal dimension of the proceeding layer. It is also normal procedure to design each size of chips for both aggregate and emulsion spread rates, then add the two oil requirements together and apply 40 percent with the first application of chips and apply the remaining 60 percent with the second application of chips.

The amount of residual asphalt required for a chip seal or seals is based on the average least dimension of the aggregate. The average least dimension is calculated from void space, gradation, and shape of the aggregate. Other factors include kind and amount of traffic and condition of the surface upon which the chip seal is being placed.

Things that can go wrong include the following:

- If any of the factors change from what was used in the chip seal design, the resultant chip seal surface will be affected.
- Other factors that influence the success or failure of the chip seal include cleanness of the chips, time of the year the chip seal is being placed, and how wet or dry the chips are when they are placed.
- The selection of the type of emulsion and any additives such as latex has an effect on the longevity of the chip seal. The emulsions with latex additives are generally considered more forgiving than emulsions without the latex additive.
- The amount of time between the first and second chip seal should not be excessive as the amount of emulsion applied with the first course of aggregate is only 40 percent of the total emulsion requirement and the aggregate is more likely to be displaced by traffic than the finished product.

ENVIRONMENTAL CONSIDERATIONS

Throughout most of the 1980s, an effort has been under way to reduce the amount of hydrocarbons as well as other pollutants in the California air and in other areas that do not comply with state or federal air quality regulations. Some air pollution control districts (frequently part of county organizations) have prohibited the use of certain asphalt cut-backs (asphalt that use volatile cutter stocks). The primary targets have been the rapid cures (RCs) and medium cures (MCs). In some instances, MCs have been allowed for prime coats, in others, no exceptions have been made. Contact should be made with the local air authority before selecting a cut-back asphalt product.

WHAT WAS LEARNED?

The thread that was consistent through a vast majority of the responses was that most LCASRs, to be successful,

1. Need to be applied to a base that is structurally adequate for the expected traffic, and
2. Need to have good drainage.

Other major recommendations are to

- Observe what is working in the geographical area of the proposed project;
- Seek out and consult with local LCASR experts;
- Find out what local materials are available and the costs;
- Customize the designs to take advantage of the local materials, economies, and environment;
- Evaluate the structural strength of the existing base and subgrade [account for traffic—use *Forest Service Handbook 7709.11*, Chapter 50 (*4*)];
- Correct any structural weaknesses with proper drainage, pit run rock, etc. (whatever is appropriate) and make bearing capacity uniform throughout the project;

- Design the LCASR on the basis of the best existing knowledge obtainable—use Asphalt Institute, FHWA, or other pertinent publications, for example, Asphalt Institute MS-14 (*1*) and MS-19 (*2*) for cold mixes, and use the design guides herein for asphalt surface treatments; and
- Be sure to design the aggregate and asphalt quantities on the basis of knowledge of them (i.e., aggregate size, shape, gradation, asphalt grades, etc.).

These techniques exist and do not need to be reinvented, just practiced.

ACKNOWLEDGMENTS

The information contained in this paper was made possible by the use of a technology transfer network on the computer communications devices of the U.S. Department of Agriculture, Forest Service, and the excellent cooperation and contributions from numerous employees.

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Pavement Management

Evaluation of a High-Resolution Profiling Instrument for Use in Road Roughness Calibration

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Response-type roughness measuring devices, now commonly used throughout the world to monitor the condition of low volume roads, require careful calibration to ensure the accuracy of their measurements. Yet there is no consensus regarding the most appropriate instrumentation for such calibration. A recent World Bank publication, reporting the findings of a series of experiments in a number of countries, proposed a hierarchy of roughness measuring instruments, the most accurate of which (termed Class I) might be used for the calibration of response-type instruments (most of which are termed Class III). Included among these Class I instruments is the Face dipstick, an inexpensive high-resolution profiling device whose features commend it for application on low-volume roads, but whose applicability for such use has not yet been properly demonstrated. By comparing two Class I profiling instruments for potential use in road roughness calibration, accepting the classification scheme established by the World Bank, it was found that the face dipstick, in its manual form, is a fast, accurate, and cost-effective alternative to other methods, including the rod-and-level method.

Low-volume roads play a critical role in stimulating economic and social activities in both developed and developing nations. Typically, these roads are constructed according to relatively inexpensive design criteria and consequently require regular maintenance to ensure structural integrity, good running surfaces, and an adequate service life. Such maintenance is equally crucial in terms of economics, because as road conditions deteriorate, total system costs dramatically increase, principally as a result of user- and vehicle-related expenses.

Unfortunately, many low-volume roads are poorly maintained. World Bank staff (*1*) have estimated that about one-third of the unpaved roads and one-quarter of the paved roads outside of urban areas in the 85 countries receiving World Bank funding now need reconstruction and that such work could cost an estimated $45 billion (1988 U.S. prices). This figure, about four times the cost of a good preventive maintenance program, represents only the agency cost of the total financial burden. What is not known is the additional economic impact on users, though it is presumed that those costs will be significantly greater when estimated with a total-cost minimizing model (*2*).

Thus, a program of preventive maintenance for low-volume roads is extremely important. And one of the key exercises within such a maintenance program is the assessing of the condition of these roads—specifically the running surface, which is commonly evaluated in terms of longitudinal (wheelpath) roughness.

Road surface roughness was known from early U.S. research (*3*) to exert a considerable influence on road-user costs; this influence has since been quantitatively investigated in a series of studies sponsored by a variety of national and international agencies (*4*). These study results now allow low-volume road planners to model the financial consequences of surface roughness deterioration with respect to both agencies and road users. Such models have stressed the link between maintenance, surface roughness, and user costs, and have focused attention on the benefits of the timely scheduling of maintenance and reconstruction. Although this is most effectively done within a pavement management system (*5*), the implementation of such a system requires a regular schedule for collecting roughness data.

There are many systems and instruments available for collecting surface roughness data, including response-type instruments, direct-profile measuring systems, indirect-profile-measuring systems, and subjective panel assessments. Not all systems have proven effective. Many are simply not designed to provide the rapid and inexpensive assessment required by the extensive networks that characterize low-volume roads. Direct profiling and indirect systems are time-consuming and costly, and subjective panel ratings are not only slow but difficult to administer and inappropriate for the primary measurement of network surface roughness. Consequently, the preferred instrument for measuring regular roughness data on low-volume roads is a response-type instrument.

All response-type instruments, including the Bureau of Public Roads (BPR) roughometer, the U.K. Transport and Road Research Laboratory (TRRL) towed fifth wheel, the TRRL bump integrator unit, and the Mays ridemeter, measure the displacement of a wheel suspension relative to a vehicle body or towing frame. The BPR roughometer is a single-wheel device towed by a vehicle; both the TRRL bump integrator unit and the Mays ridemeter are typically mounted in the rear of a light vehicle, usually an automobile, and measure the cumulative movements of the rear axle relative to the vehicle body. Data produced by vehicle-mounted systems are affected by several factors other than surface roughness, including vehicle speed, deteriorating suspension systems, tire pressure variations, uneven tire wear, and vehicle mass. In the Brazil study (*6*), a 5 percent change in vehicle mass or a 1 percent change in tire pressure altered the response count by around 2 percent. Therefore, if pavement management systems for low-volume roads are to be effective, these response-type

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measuring devices must follow regular calibration procedures to ensure the accuracy and reliability of reported data.

Although this calibration requirement was comprehensively addressed in guidelines developed by Sayers et al. (*7*), earlier researchers had been following distinct calibration routines in their studies. For example, the Brazil study used nine Maysmeter-equipped vehicles that were regularly calibrated over 20 control sections for which a reference roughness standard had been determined (an activity that consumed up to 20 percent of available instrument time). Elevation data from longitudinal profiles were measured using a K. J. Law profilometer; the data were then processed through a quarter-car simulation of the BPR roughometer to give a quarter-car index (QI), or average rectified slope expressed in counts per kilometer. This statistic is similar to the recently recommended International Roughness Index (IRI) (*8*). For the majority of agencies, however, the use of a profilometer is not feasible because of its cost, maintenance requirements, and the need for highly trained staff. Accordingly, other instruments for determining low-cost reference roughness standards need to be investigated.

A hierarchy of such instruments was reported in World Bank guidelines (*7*), categorized by Paterson (*9*), and adopted in the recent FHWA requirements for roughness data collection (*10*). These instruments are presented in Table 1, which shows that the highest profile resolution is provided by three devices (termed Class I). Using this categorization, researchers at the Center for Transportation Research (CTR) at the University of Texas at Austin investigated the feasibility of implementing a high-profile resolution method for the Texas State Department of Highway and Public Transportation (SDHPT). These results are of interest to highway agencies responsible for low-volume road management and contemplating using response-type devices requiring regular calibration against a high-resolution profiler.

A high-resolution Class I profiling instrument is evaluated for use in road roughness calibration of response-type instruments, comparing specifically the Face dipstick profiles with rod-and-level profiles (the TRRL beam was not available for evaluation). The dipstick and its operation are presented first, including a summary of features, an evaluation, and recommended checks. Next, the dipstick is evaluated in both its automatic-data-transfer mode and in its manual-transfer mode. Two dipsticks were used in all evaluations, enabling instrument-to-instrument variations to be determined. The dipstick's elevation output was compared also with the output

TABLE 1 CLASSIFICATION OF PROFILING INSTRUMENTS

CLASS	METHOD	MAX. ERROR	MEASUREMENT INTERVAL	EXAMPLE INSTRUMENTS	COMMENTS
I PRECISION PROFILES	MANUAL ABSOLUTE ELEVATION RELATIVE TO TRUE HORIZONTAL	1.5% BIAS; 19 INCH/MILE	LESS THAN OR EQUAL TO 1 FOOT	ROD & LEVEL DIPSTICK TRRL BEAM	DATA COLLECTED MANUALLY AND PROCESSED TO GIVE ROUGHNESS STATISTIC; VERY ACCURATE BUT LABORIOUS AND SLOW
II DIRECT PROFILING MEASUREMENT	ELECTRONICALLY MEASURED ELEVATION PROFILE FROM ARTIFICIAL "HORIZONTAL" DATUM	5% BIAS; 44 INCH/MILE	LESS THAN OR EQUAL TO 2 FEET	GM PROFILOMETER K.J. LAW PROFILOMETER TEXAS PROFILOMETER FRENCH APL	DATA COLLECTED FROM MOVING VEHICLE; DIFFER IN REFERENCE USED AND METHOD OF SENSING; NOT ABSOLUTE PROFILE BECAUSE OF LACK OF LOW FREQUENCY RESPONSE; CAN BE UTILIZED FOR CALIBRATION
III A RTRRMS	MEASURE DYNAMIC RESPONSE OF A MECHANICAL DEVICE TO ROADWAY SURFACE	10% BIAS; 32-63 INCHES/MILE	CONTINUOUS OVER TEST SECTION LENGTH	MAYS RIDE METERS ARAN BPR ROUGHMETER DYNATEST 5000 RDM COX ROADMETER	MOST COMMON INSTRUMENTS; (1) MEASURE OF AXLE-BODY MOVEMENTS USUALLY SUMMED TO GIVE CUMULATIVE "BUMPS" PER UNIT DISTANCE, OR (2) MEASURES ACCELERATIONS OF AXLE OR BODY VIA ACCELEROMETERS; DATA COLLECTED AT HIGHWAY SPEEDS; REQUIRES CALIBRATION
III B MOVING DATUM PROFILES	MEASURE DEVIATION OF PROFILE RELATIVE TO A DATUM MOVED ALONG ROAD SURFACE	BLANKING BANDS USED TO FILTER OUT CONSTRUCTION TECHNIQUES SUCH AS TINING	CONTINUOUS OVER TEST SECTION LENGTH	ROLLING STRAIGHT EDGE SLIDING STRAIGHT EDGE PROFILOGRAPHS	INSENSITIVE TO WAVELENGTHS EQUAL TO INSTRUMENT BASELENGTH, PROFILOGRAPH AVERAGES END REFERENCE POINTS; SIGNAL GAIN HIGHLY TUNED AND VARIABLE (IDEAL IS UNIFORM GAIN)
IV	SUBJECTIVE ESTIMATES MADE BY OBSERVER(S) USING A DESCRIPTIVE SCALE				NOT SUITABLE FOR COLLECTING ROUGHNESS DATA FOR HPMS

from the rod-and-level surveys on several pavement sections. The elevations from both the dipstick and rod-and-level were used to calculate a number of roughness statistics in Texas. These statistics allow comparisons of roughness calculations on the basis of the data from the two survey techniques. Finally, recommendations are made concerning the use of the face dipstick as a high-resolution profiling instrument for use in calibrating response-type roughness devices.

OPERATION OF THE DIPSTICK

The dipstick, originally designed by the Face Company as a manually read instrument for evaluating the flatness of concrete floors, employs an inclinometer to determine the difference in elevation between its two feet (spaced 1 ft apart). This inclinometer acts as an electronic pendulum for determining and displaying the elevation differences. Providing readings to 0.001 in., the dipstick, according to its manufacturers, is accurate to ±0.0015 in. per reading. Although the manual-read version requires two persons for proper operation (one person to move the dipstick down the survey line, the other to record the readings), the auto-read version of this instrument requires only one (readings are automatically read, captured, and stored by the onboard computer). By eliminating the requirement for a second operator, the auto-read version reduces both the data transcription time and the possibility of transcription errors.

The computer onboard the auto-read dipstick uses an electronic interface built into the inclinometer circuitry, with a PC-2 installed on the dipstick's handle (Figure 1). The elevation data are captured and stored on the PC-2 for later transfer to an IBM-compatible computer by an RS-232 communications port. Once the raw elevation data are transferred to the IBM-type computer, a set of application programs is used to view, manipulate, or process the data into roughness statistics, one of which is the IRI.

In using the dipstick, an operator manually walks down a wheelpath (previously marked in some way to guide the operator). The operator rotates the dipstick from one elevation location to next, leaving the front foot of the dipstick on the pavement surface while the back foot is rotated forward. If the front foot is lifted from the pavement, the reference elevation will be lost and the procedure must be started over. In the manual version of the dipstick, as mentioned earlier, a second operator (transcriber) records the elevation readings displayed on the dipstick's display windows (Figure 1). The operator waits until the display has settled and then calls out the reading (displayed on the forward-pointing foot) for the transcriber to record. In the automatic version, the elevations are captured automatically on the PC-2. A series of beeps from the PC-2 indicates to the operator that the reading has been taken and that the computer is ready for the next location's reading.

The operation of both auto-read and manual-read dipstick has several drawbacks that can adversely affect the data collected. For example, the leveling of the dipstick's body can be a tedious process in situations where a smooth surface is unavailable. If the dipstick's feet are not level to within ±0.003 in., as prescribed by the manufacturer, the cumulative error from reading to reading becomes significant. Calibration is

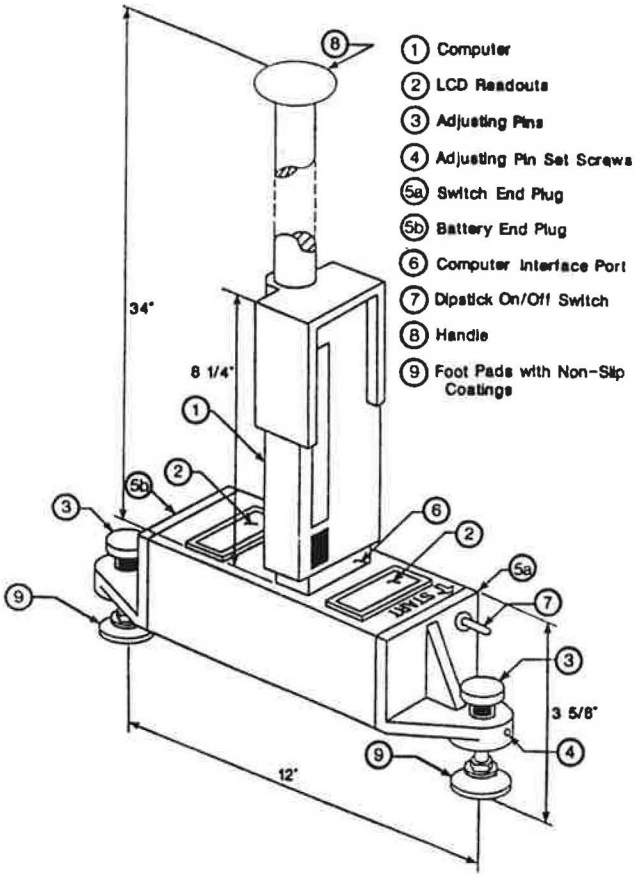

FIGURE 1 Auto-read version of the dipstick.

another problem. Although the calibration of the inclinometer can be checked with the 0.125-in. gauge block included with the dipstick, if the calibration check fails, the user must return the dipstick to the manufacturer for proper adjustment.

Additionally, the dipstick's feet and foot pads can cause cumulative errors during a survey run. In particular, the ball-and-socket joints that allow the dipstick's feet to rotate smoothly have been a source of problems. These joints should be kept as clean as possible and lubricated with a heavy-duty oil (e.g., SAE 30-WT); with proper lubrication, the feet will rotate freely and will not trip as the dipstick is rotated and placed for the next reading. Also, the adhesive used to cement the rubberized pads to the feet has tended to debond. When this occurs during a survey run, it can cause errors in the data. The pads themselves have come apart during multiple runs on newly constructed rigid pavements, a result of the rough microtexture placed on new rigid pavements.

Problems relating to the battery compartment on the dipstick's body can also result in data errors. If the compartment cover is not tightly pressed against the dipstick's body after the batteries are changed, a small space is created, which can cause distance errors during a survey run. For example, a space as small as 1/32 in. can cause a distance error of 33 ft over a 1,056-ft survey section.

Finally, there is no low-battery indicator on the dipstick. Experience with this instrument has shown that battery life is only 6 to 8 hr. If batteries fail in the middle of a survey

section, the section must be reevaluated. Therefore, it is recommended that the batteries be changed daily.

TEST SITE DESCRIPTION

Several test sites were used for evaluating both the auto-read and the manual-read versions of the Face dipstick. All sites were laid out using the same procedures, though some of the sites had, in addition, a rod-and-level survey performed for comparison. A detailed description of the test sites and their locations was given in a recent CTR Technical Report (*11*). The following presents the identification codes used for each test site and describes how each test site was laid out for the evaluation effort.

The initial evaluation of the prototype auto-read dipstick was performed on several asphaltic pavements located at the Balcones Research Center (BRC) of the University of Texas in Austin. These sites were of varying lengths and all had rod-and-level surveys performed at 2-ft intervals. All but one of these sites were used primarily for demonstration purposes, and the one site referred to is designated BRC and is 1,000 ft long.

The La Grange test site, selected to determine the dipstick's response on the rough microtexture on new rigid pavements, consisted of newly constructed concrete pavement on a bypass around the city of La Grange, Texas. Two 1,000-ft sections, each consisting of the travel lanes in each direction, were marked. Although the section was rod-and-leveled, a number of high-speed roughness instruments from the Texas SDHPT were run on the section for comparison. These instruments included the Class II profilometer and several Class III instruments. The Class III instruments consisted of the Highway Products International automatic road analyzer (HPI ARAN), a Maysmeter, a Texas SI-ometer, and the McCracken California-type profilograph.

Another extensively used site was a 200-ft strip of a city street in Austin, Texas, that was rod-and-leveled at 6-in. intervals. The low traffic volume (traffic control consisted of a single flagman) permitted repeat runs to be performed and ensured that the rod-and-level survey results would remain relatively constant over time. This site is hereafter referred to as the Oakmont test site.

Additional test sites included a number of asphaltic county and state roadways around the Austin area. These sites had been used for a number of years by the Texas SDHPT as calibration sections for its high-speed roughness instrumentation (*12*). All of these sections were 0.2 mi long and some had been surveyed using the rod-and-level. (These sections are referred to as "Austin Test Section," or ATS, followed with a discrete number as an identifier.) The majority of these sites required lane closure (with the associated traffic control), while some required flagmen only. The ATS sites were particularly useful in that they exhibited road surfaces ranging from very smooth to very rough.

Each of the wheelpaths in every test site except Oakmont was marked using the same technique. A start location was first painted on the pavement surface across each travel lane. In determining the individual wheelpaths within the travel lane, an assumed 65-in. wheelbase width [established by the Strategic Highway Research Program (SHRP) so as to facilitate comparison with the profilometer, which has a 65-in. wheelbase width] was centered in the travel lane, with a string line stretched down both wheelpaths of each travel lane. A series of dots painted on the pavement surface along the string lines allowed the dipstick operators to follow approximately the same wheelpath during repeat runs. The distance of each test section was determined using a steel tape, with the end of the section designated by a stop line painted across the travel lane. The Oakmont section was marked using the same procedure except that the two wheelpaths were centered about the center of the entire pavement width.

AUTO-READ DIPSTICK EVALUATION

The purpose of the initial auto-read evaluation was to determine whether this dipstick version is reliable and repeatable as a Class I field instrument. Although there were other issues that presented themselves during the evaluation process—issues that were briefly described above and that are reviewed elsewhere (*11*)—only the major concerns with the performance of the auto-read dipstick are addressed.

Two dipsticks were used in this evaluation effort. One was purchased by CTR staff for the Texas SDHPT and is referred to as the UT dipstick; the other was purchased for the SHRP program (SHRP was interested in using the dipstick for profilometer calibration and substitution), and will be referred to as the SHRP dipstick.

Problems in the operation of the auto-read version of the dipstick included situations where the dipstick either (a) failed to take a reading or (b) took a reading that should not have been taken. The first situation will be referred to as "no readings"; the second situation will be referred to as "false readings."

No Readings

Occasionally, the dipstick fails to take readings. When this occurs, the screen does not go blank and the beep does not sound, even after the operator has rotated the dipstick to take a new reading. The operator must constantly look at the dipstick's display screens to determine if a reading has been taken. When the display screen blanks and a number reappears, the dipstick has settled and is ready for a new reading. When it is discovered that a reading has not been taken and the reference has not been lost, the operator can lean the dipstick's body to cause the instrument to take a reading. If the operator fails to detect a no-reading situation, the instrument will, for the remainder of the survey, read the wrong foot. This results in the loss of the reference elevation, and consequently the remainder of the elevations will be opposite in sign.

False Readings

False readings represent the most serious problem in the auto-read version of the dipstick. False readings can cause the reported direction of the pavement's design slope to change several times during a single survey run. Figures 2–4 show

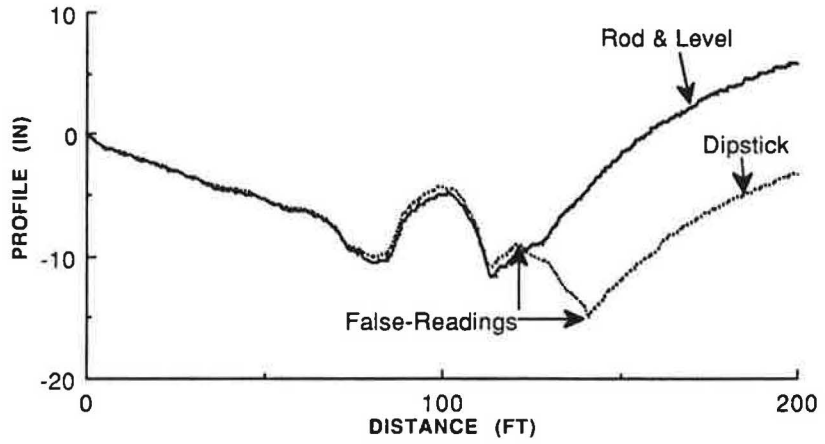

FIGURE 2 Dipstick data with false readings versus rod-and-level survey.

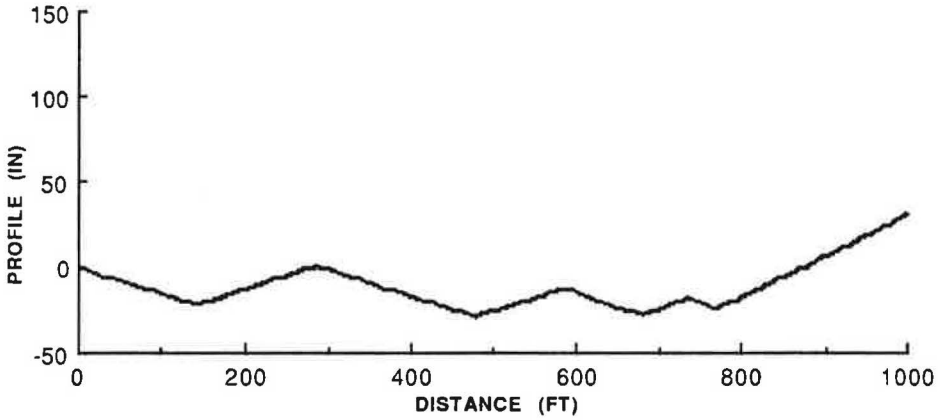

FIGURE 3 Dipstick data with false readings.

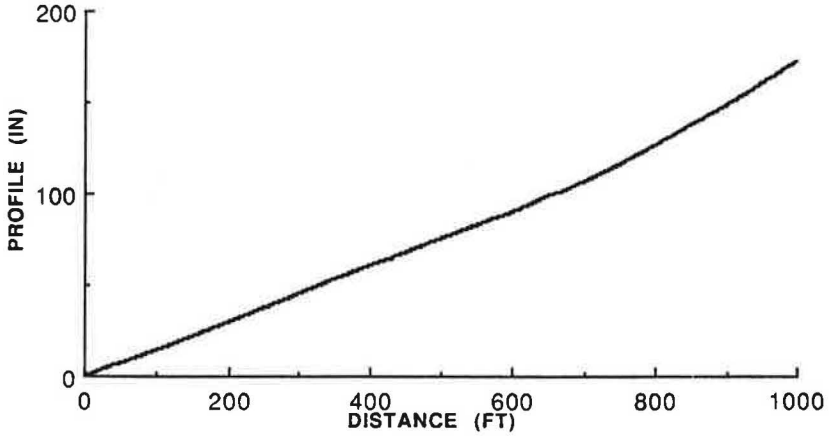

FIGURE 4 Profile of surface shown in Figure 3 with proper slope.

the false-reading situation, representing the running sum (the section profile with the design slope intact) of the raw elevation data captured on a survey section. Figure 2 shows the dipstick evaluation of the Oakmont test site (previously rod-and-leveled), with the two false readings labeled. These false readings caused the sign on all subsequent readings to be opposite, yielding results that were clearly erroneous. Another example of false readings can be seen in Figure 3, where seven false readings are shown. Compounding this problem was the fact that the new acquisition software, updated by the Face Company, reads first the foot opposite the one that the earlier version read first. Because the operator was unaware of this update, the initial elevations are opposite in sign. Figure 4 is the actual running-sum profile of the La Grange test section in Figure 3.

The most reliable and consistent auto-read Dipstick data are collected when there is more than one dipstick available to take readings along parallel wheelpaths and when the operators are able to constantly check each other's readings and orientation. Such an arrangement can make use of the edit software on the acquisition program to collect a reliable set of data, and, in addition, is psychologically helpful to the dipstick operators. It takes an experienced operator 80 to 90 min to dipstick 1,000 ft. At this pace, an operator's mind tends to wander, resulting in less attention being paid to the job of dipsticking. Consequently, the use of a single operator increases the likelihood that a false reading or a no reading will go unnoticed.

MANUAL-READ DIPSTICK EVALUATION

The manual-read version of the Face dipstick was evaluated as an alternative to the rod-and-level survey after the auto-read version was determined to have problems with repeatability and reliability. The auto-read version of the dipstick can be used in the manual mode by reading and recording the LCD displays located on the dipstick's case (Figure 1). The difference in sensitivity between the SHRP and UT dipsticks could still be a problem in the manual mode of operation. The manual-read evaluation concentrated on determining whether operation of different dipsticks showed different results. In addition, the manual-read version of the dipstick was compared with rod-and-level surveys on three different sites. These comparisons included raw elevations, running-sum differences, and calculated roughness statistics.

The operation of the manual-read version of the dipstick is the same as described in the auto-read version evaluation above. The same manufacturer's recommended leveling and calibration check procedures were followed. The dipstick's feet were kept well lubricated and the batteries were changed daily as precautions against erroneous data and abandoned runs. The data were read by one operator and recorded by a second crew member onto a form. The audio-tape method of recording the elevation data was also evaluated as an alternative to having the two-man crew usually associated with the operation of a dipstick.

This testing was concerned with comparing two different instruments on the same pavement sections. The repeatability of the same instrument on the same pavement sections was evaluated, along with the time needed to manually record the data and transcribe it onto a computer. The comparisons with the rod-and-level survey were based on the raw elevation data obtained from both instruments, along with the running-sum differences calculated from these data. Several roughness statistics were calculated from the data and comparisons were made between dipstick and rod-and-level interpretation of the pavement surface profile.

UT Dipstick Versus SHRP Dipstick

The manual-read versions of the UT and SHRP dipsticks were evaluated on three different test sites at three different lengths, with the data collection and transcription processes analyzed for time consumption. Two different methods of collecting the data were used during this process. The first method required a two-person crew (the dipstick operator would walk the dipstick down the wheelpath to be evaluated while reading and calling out the elevation numbers displayed on the dipstick's LCD windows; the other crew member would manually write the elevation data on a form for later transcription). The second method required a single operator only. A microphone attached to the operator's lapel and to an audiotape recorder was used to tape the elevation data.

The two data collection methods took approximately the same amount of time. The survey of the 500-ft BRC test site averaged 39.3 min, whereas the survey of the 200-ft Oakmont site averaged 15.6 min and the 1,056-ft ATS 04 averaged 83 min. The average numbers of readings per minute for the data collected at the sites were as follows: the BRC site, 12.8; the Oakmont site, 12.8; and the ATS 04 site, 12.7.

The audiotape-recorded data were transcribed directly to the computer and onto a recording form. This process not only provided a mechanism for determining if significantly more time was needed to obtain a hard-copy of the raw data; it also could determine if significant translation errors occurred. The transcription of the raw taped data to the computer or to the recording forms took approximately the same amount of time as required for running the dipstick through the test site. Because the tape was running continually during the acquisition of the elevation data, it took the transcriber the same amount of time to listen to the tape as it did to run the section. Obviously, transcribing the data from the tape to the recording form then to the computer added both time and a greatest risk of transcription error, but it was considered prudent to have a hard copy of the raw data. The biggest problem with the tape recording method was traffic noise. Several of the taped readings were unintelligible because of the noise, and required operators to decide whether to recollect or interpolate the data from the surrounding elevation points.

The data that were hand recorded and then transcribed into the computer required less input time than the taped data, with the time differential approximately one-half that of the actual data acquisition for each test site length. The more proficient the transcriber became at entering data into a particular computer program, the faster and more reliable the data transcription became, though obviously, there is a limit to the speed at which a person can input data into a computer.

While the biggest disadvantages of hand-recording the raw data are the necessity of a two-person data collection crew and the greater expense that goes with it, these disad-

vantages are offset by the advantages of hand recording, rather than tape recording, the data. The transcription of the tape-recorded data requires about twice the time it takes to transcribe the handwritten data. If a hard copy of the raw data is desirable, then the tape-recorded data must be transcribed twice, requiring more transcription time and increasing the risk of transcription error. Regardless of the recording method being used, the transcribed data must be checked before it can be considered reliable. The problem of trying to use data made unintelligible by traffic noise is the most serious disadvantage of the audiotape data recording method. If accuracy and reliability of the data are of paramount importance, then the handwritten recording method is recommended. The more costly tape recording method could be made reliable if traffic noise could be eliminated from the collected data.

The dipstick data should be verified by closing the loop, that is, using a methodology that ensures that a reliable set of data representing the true surface profile is obtained. A test site should be evaluated in one direction from beginning to end. Being careful not to life the dipstick from the surface, the operator should reevaluate the test site by walking the dipstick in the opposite direction down the same wheelpath (a process that provides the evaluator with a second set of data for comparison). The raw data and the running sums of the data can be plotted to find differences in the forward and reverse runs of the same wheelpath. Each individual dipstick has different responses to the same surface but these differences are in the 0.01-in. range. This magnitude of error is less than that associated with placing the dipstick's feet on exactly the same spot on the pavement surface during repeat runs. If transcription or dipsticking error is in the data sets, relatively large spikes or running sum differences will be found during the two runs. In this case, a third run would be required.

Dipstick Versus Rod-and-Level

Three test sites were used to compare the rod-and-level survey interpretation of the pavement surface profile with that of the manual-read version of the dipstick. In terms of the recently instituted IRI, these sites exhibited paved-road roughness ranging from 85 to 350 in./mi. They were marked with a series of guide dots every foot from the beginning to the end of each wheelpath in a travel lane.

The rod-and-level crew were instructed to survey 200 ft at a time before moving the instrument to the next setup location. No benchmarks were established. The crew would take readings up one wheelpath and down the other wheelpath. The relative changes at the end of every 200-ft section were checked by reshooting the last 10 ft of both wheelpaths after moving to the next section. The elevations were read to the hundredth of a foot and estimated to the thousandth of a foot. A rod with a vernier was chosen to provide this survey with the best estimate.

The survey crew consisted of three people: one who ran the instrument and called out the readings, another who set the rod in position and made certain it was plumb while the readings were being taken, and a third person who adjusted the vernier and wrote the readings on the reporting form. The actual survey of each 0.2-mi section took the crew approximately 2 days to complete. Then the data had to be entered into the computer for comparison and analysis, a process that took approximately 4 hr per wheelpath. Because the survey was conducted in 200-ft intervals, the readings for each 200-ft section started with different elevations. This meant that the data had to be adjusted so that the relative elevation differences were accurately carried through the transition section. Repeating the last 10 readings of each 200-ft section helped the data transcriber make the proper adjustment to the elevation data (i.e., because the instrument was moved every 200 ft, the 10 readings served as a closure from one site to the next).

The transcribed data from the dipstick and the rod-and-level survey were plotted so that the raw elevations and the running sums could be compared. The rod-and-level survey data were reported and transcribed with feet as the units (the dipstick data are reported in inches). To make the necessary comparisons, the rod-and-level data were converted to units of inches by multiplying the rod-and-level data by 12. Some of the rod-and-level survey data contained spikes, which meant that the process of multiplying by 12 magnified these spikes. The data transcriber located these spikes on the raw data forms and then attempted to determine if the data were transcribed properly or if the transcribed data actually represented what was written on the form. A determination was made whether the spiked data represented a hole or a rock in the road surface (in which case the data would be thrown out and replaced by averaging the elevations before and after the errant data). This situation was never encountered. The spiked data seem to represent either the instrument operator's reading mistake or the data being wrongly recorded on the form.

Figures 5 and 6 show the problem rod-and-level spikes cause in comparing the dipstick data. Figure 5 shows the dipstick elevations taken from the outside wheelpath of ATS 36, while Figure 6 shows the rod-and-level elevations of the same roadway. Although the scale of the y-axis on both figures is in inches, the ranges are different on the two figures. The range on Figure 5 is from 0.4 to -0.8 in., while the range on Figure 6 is 2 to -2 in. The different ranges were caused by spikes encountered in the rod-and-level data. In contrast, if the plots of the inside wheelpath of ATS 36 are viewed, the ranges of

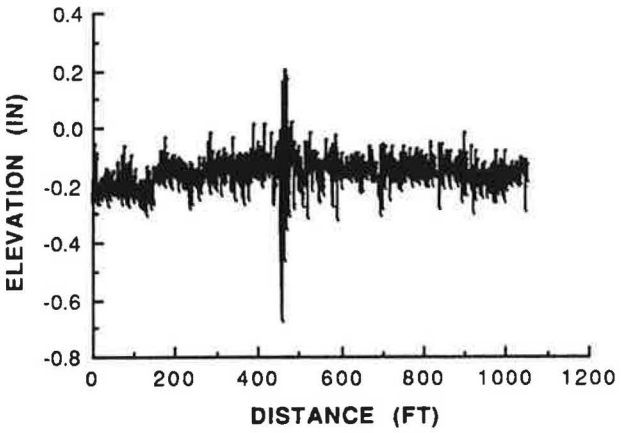

FIGURE 5 Dipstick elevations from ATS 36 outside wheel path.

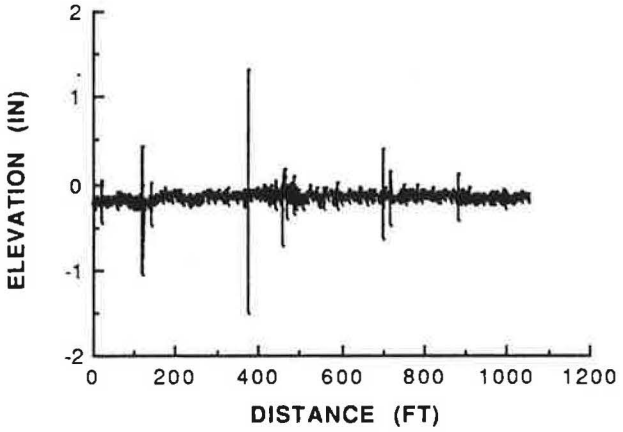

FIGURE 6 Rod-and-level elevations of ATS 36 outside wheel path before corrections.

the y-axis are the same. Figures 7 and 8 show plots of the dipstick and the rod-and-level data from the inside wheelpath of ATS 36, respectively.

The spikes associated with rod-and-level data do not necessarily translate into large running sum differences when compared to the dipstick data. Figure 9 shows the running sums of both the rod-and-level survey and the dipstick data from the outside wheelpath on ATS 36 overlaid on the same plot. As indicated, the difference between the last readings of each instrument is 1.608 in. Figure 10 shows the running sums of the inside wheelpath on ATS 36. The difference between the last readings is 3.605 in. As seen in Figure 5, many large spikes occurred in the raw elevations of the rod-and-level data from ATS 36 outside wheelpath. Most of these spikes seem to be opposite in magnitude, offsetting each other when the running sum is calculated, producing smaller differences when compared to the dipstick data. A comparison of Figures 7 and 8 indicates that the dipstick and rod-and-level elevations are similar, yet the difference in the running sums is 3.065 in.

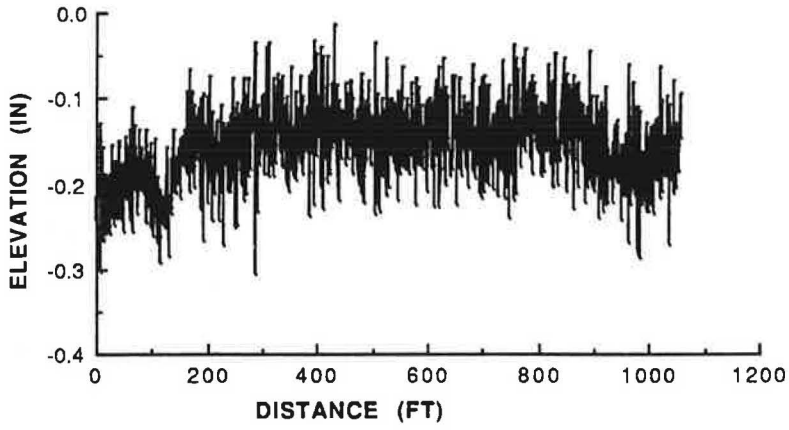

FIGURE 7 Dipstick elevations from ATS 36 inside wheel path.

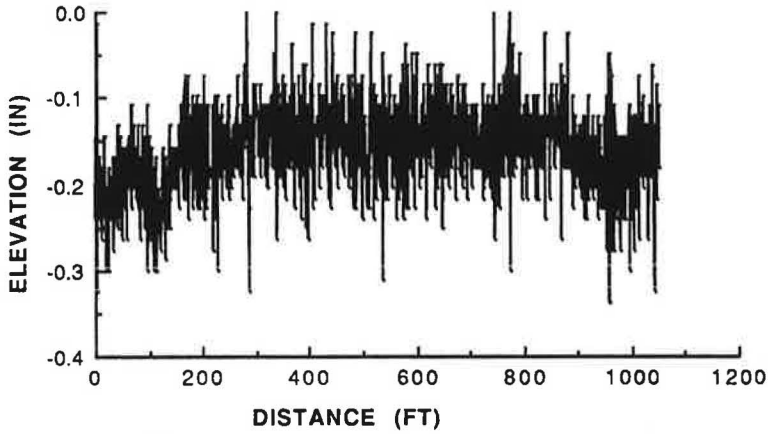

FIGURE 8 Rod-and-level elevations from ATS 36 inside wheel path.

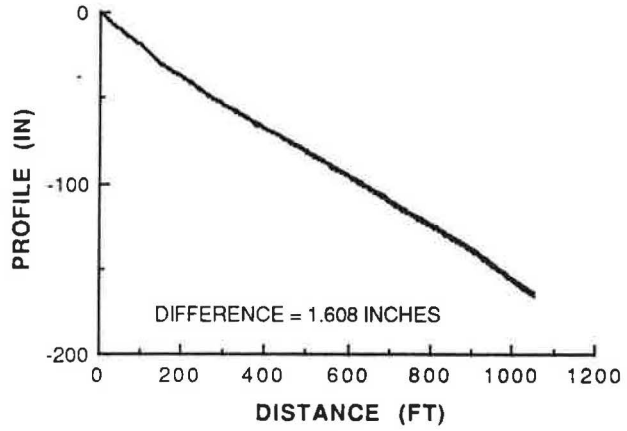

FIGURE 9 Profile comparison of dipstick and rod-and-level on outside wheel path of ATS 36.

Roughness Statistics Comparisons

There are several roughness statistics that can be used to evaluate the surface roughness characteristics of the pavement. These statistics (as against raw profile data) are necessary for the calibration of response-type equipment. The correlation between the calculated values from Class I instruments are regressed against the output of an Individual RTRRM system. For this evaluation, the IRI, root mean square vertical acceleration (RMSVA), serviceability index (SI), and a simulated Maysmeter output (MO) were calculated. The IRI and RMSVA calculations are based on the raw elevation data from the dipstick and the rod-and-level survey. These statistics were calculated for each wheelpath in a test section. In addition, the dipstick was run in the reverse direction on each wheelpath, generating two sets of IRI and RMSVA statistics per wheelpath. The SI and MO statistics are based on the 4- and 16-ft wavelengths of the RMSVA statistic. The calculations for these wavelengths are averaged for both wheelpaths of a test section and the average is used for the calculations (*13*).

Tables 1–5 present the summary statistic results from the dipstick and rod-and-level surveys performed on ATS Sections 42, 36, and 04. These sections were chosen for comparison because they represented a range of pavement surface roughness. The areas where spikes were noticed in the rod-and-level data had to be adjusted, and the process for adjusting this data involved averaging the data points before and after the spike and using this value as the adjusted data point. The shorter wavelengths from the rod-and-level survey data were affected to the greatest degree, even after the data were adjusted. Tables 4–6, which compare RMSVA by wavelength, indicate that, as the wavelength increases, the better the dipstick and the rod-and-level compare.

CONCLUSIONS AND RECOMMENDATIONS

Where response-type roughness devices are used to monitor the condition of low-volume highway networks, there is a distinct need for a high-resolution profile instrument to maintain paved calibration sections. Typically, such sections are, as described in this study, on paved highways having a full range of roughness and where truck usage is low. The relatively short length of calibration sections—normally 0.2 mi—makes it feasible to employ labor-intensive methods of profile determination. Rod-and-level methods can be used (*14*) but are expensive, slow, and, consequently, troublesome. The dipstick appears to be a workable alternative to rod-and-level methods, and the CTR study has demonstrated its potential. The general conclusions are as follows:

1. The manual-read version of the dipstick is an effective Class I profiling instrument, as long as operational techniques, including loop-closing and repeat runs, are followed. It is many times faster than the rod-and-level surveying method, and has a resolution 12 times better, making it an extremely accurate device.
2. The 1989 auto-read version of the dipstick was determined to be unsuitable as a Class I profiling instrument. In particular, the device frequently failed to record a change in elevation and, in addition, was susceptible to false readings;

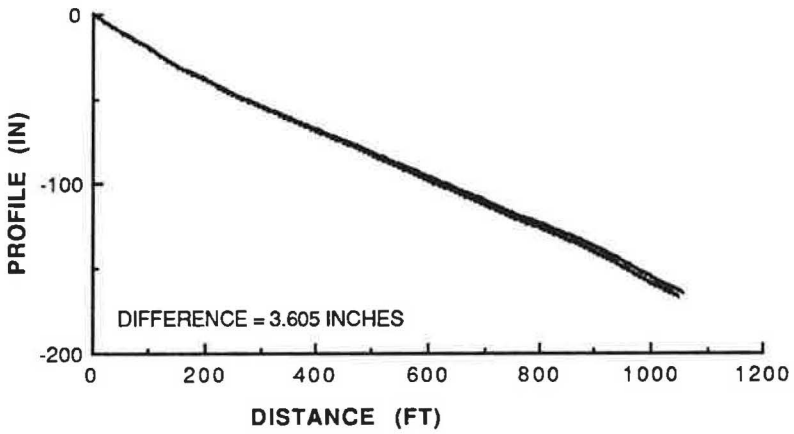

FIGURE 10 Profile comparison of dipstick and rod-and-level on inside wheel path of ATS 36.

TABLE 2 DIPSTICK VERSUS ROD-AND-LEVEL IRI STATISTICS

		DIPSTICK	ROD-AND-LEVEL
ATS Number	Wheel Path[1]	IRI (in/mile)	IRI (in/mile)
04	IS(F)	280.2	280.16
	IS(R)	277.8	
	OS(F)	352.3	352.39
	OS(R)	349.4	
36	IS(F)	97.4	112.03
	IS(R)	101.0	
	OS(F)	154.2	158.74
	OS(R)	142.7	
42	IS(F)	88.7	95.46
	IS(R)	91.7	
	OS(F)	93.6	99.42
	OS(R)	93.9	

[1]IS(F) indicates inside wheel path forward direction, IS(R) indicates reverse direction, OS(F) indicates outside wheel path forward direction, OS(R) indicates reverse direction.

TABLE 3 DIPSTICK VERSUS ROD-AND-LEVEL SI AND MO STATISTICS

		DIPSTICK		ROD-AND-LEVEL	
ATS Number	Wheel Path[1]	SI	MO	SI	MO
04	IS	1.365	194.82	1.34	196.67
	OS	1.357	194.82		
36	IS	3.26	72.87	3.26	72.79
	OS	3.34	69.56		
42	IS	4.068	40.75	3.88	47.57
	OS	4.089	40.01		

[1]IS indicates inside wheel path, OS indicates outside wheel path.

TABLE 4 DIPSTICK VERSUS ROD-AND-LEVEL RMSVA STATISTICS FROM ATS 04

		DIPSTICK	ROD-AND-LEVEL
Wheel Path[1]	Wavelength (feet)	RMSVA	RMSVA
IS	1	32.88	38.09
	2	13.47	14.35
	4	6.15	6.02
	8	2.85	2.80
	16	0.96	1.03
	32	0.39	0.39
	64	0.15	0.14
	128	0.05	0.05
OS	1	35.81	59.63
	2	17.76	18.66
	4	8.42	8.86
	8	3.08	3.00
	16	1.19	1.10
	32	0.47	0.46
	64	0.21	0.21
	128	0.06	0.06

[1]IS indicates inside wheel path, OS indicates outside wheel path.

TABLE 5 DIPSTICK VERSUS ROD-AND-LEVEL RMSVA STATISTICS FROM ATS 36

		DIPSTICK	ROD-AND-LEVEL
Wheel Path[1]	Wavelength (feet)	RMSVA	RMSVA
IS	1	29.00	35.11
	2	8.31	9.94
	4	2.30	2.25
	8	0.77	0.77
	16	0.34	0.31
	32	0.20	0.20
	64	0.09	0.08
	128	0.04	0.04
OS	1	35.39	42.08
	2	12.15	14.23
	4	4.45	4.46
	8	1.05	1.17
	16	0.52	0.56
	32	0.21	0.20
	64	0.07	0.06
	128	0.03	0.03

[1]IS indicates inside wheel path, OS indicates outside wheel path.

TABLE 6 DIPSTICK VERSUS ROD-AND-LEVEL RMSVA STATISTICS FROM ATS 42

		DIPSTICK	ROD-AND-LEVEL
Wheel Path[1]	Wavelength (feet)	RMSVA	RMSVA
IS	1	13.86	21.18
	2	4.67	6.14
	4	1.95	2.20
	8	0.89	0.95
	16	0.41	0.41
	32	0.22	0.22
	64	0.12	0.12
	128	0.08	0.08
OS	1	16.84	21.67
	2	5.63	6.97
	4	2.08	2.42
	8	0.91	0.99
	16	0.36	0.37
	32	0.21	0.19
	64	0.13	0.12
	128	0.08	0.07

[1]IS indicates inside wheel path, OS indicates outside wheel path.

both problems pose severe drawbacks in terms of instrument repeatability and reliability. Even the use of two dipsticks on the same section did not entirely overcome these problems and, thus, the method did not represent a feasible operation. The manufacturer has been provided with a list of recommended design, hardware, and software changes the CTR staff consider necessary for upgrading this to a reliable Class I device. If the manufacturer incorporates these recommendations into a new model, the modified version should be fully evaluated before field use.

3. Both versions of the dipstick showed instrument-to-instrument differences in sensitivity that are believed to result from variations in production quality control related to the internal inclinometers. The manufacturer is aware of this problem, and current versions may provide more consistent inclinometer performance. Nevertheless, a user should check the calibration of each new dipstick and evaluate its performance against a rod-and-level survey, or against a proven dipstick before field use.

The need to calibrate response-type roughness measuring devices against high-resolution profiles of paved highway sections can therefore be addressed by using the manual dipstick. The instrument, when properly calibrated and operated, can give profiles as good as those from rod-and-level surveys at a fraction of the time and cost. Guidance on appropriate calibration and operation has been reported and can be supplemented by more detailed examination of a CTR report (*15*). Those responsible for the effective management of low-volume road systems, where response-type devices are employed to monitor surface condition, can use the manual dipstick as an integral part of the calibration process. Properly calibrated instruments will produce condition data that can be employed to enhance the effectiveness of the increasing number of low-volume pavement management systems now being instituted throughout the world.

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MERLIN—A Low-Cost Machine for Measuring Road Roughness in Developing Countries

M. A. CUNDILL

The roughness of a road's surface is an important measure of road condition and a key factor in determining vehicle operating costs. A simple roughness measuring machine has been designed especially for use in developing countries. It is called MERLIN (Machine for Evaluating Roughness using Low-cost INstrumentation). The device can be used either for direct measurement or for calibrating response-type instruments such as the vehicle-mounted bump integrator. It consists of a metal frame 1.8 m long with a wheel at the front, a foot at the rear, and a probe midway between them that rests on the road surface. The probe is attached to a moving arm at the other end of which is a pointer that moves over a chart. The machine is placed at successive locations along the road and the positions of the pointer are recorded on the chart to build up a histogram. The width of this histogram can be used to give a good estimate of roughness in terms of the International Roughness Index. Calibration of the device was carried out using computer simulations of its operation on road profiles measured in the 1982 International Road Roughness Experiment. The MERLIN is in use in a number of developing countries. It can usually be made locally at a current cost of typically U.S. $250.

The longitudinal unevenness of a road's surface (normally termed its roughness) is both a good measure of the road's condition and an important determinant of vehicle operating costs and ride quality. Within developing countries, there is particular interest in the effect on vehicle operating costs. A number of studies [e.g., by Hide et al. (*1*), Hide (*2*), the Central Road Research Institute (*3*), and Chesher and Harrison (*4*)] have shown how roughness can influence the cost of vehicle maintenance, the rate of tire wear, and vehicle running speeds (and hence vehicle productivity).

Reliable measurement of road roughness is therefore seen as an important activity in road network management. A variety of machines have been developed to make these measurements, and a number of roughness scales have been established.

However, despite the range of different measuring machines that exists, it was believed that there was a need, particularly within developing countries, for a different type of device. Ideally, it should be simple, inexpensive, require no calibration, and be able to make fairly rapid measurements of reasonable accuracy on one of the standard roughness scales. Such a machine might be used either directly to measure roughness or for calibrating other roughness-measuring equipment, particularly the widely used vehicle-mounted bump integrator.

The standard roughness scale that has been used for many years by the Overseas Unit of the U.K. Transport and Road Research Laboratory (TRRL) in its studies on vehicle operating costs and pavement deterioration is the output of the fifth-wheel bump integrator (BI) towed at 32 km/hr. However, another scale that is now being widely used is the International Roughness Index (IRI) (*5*). This scale, which is derived from road profile data by a fairly complex mathematical procedure, represents the vertical movement of a wheel with respect to a chassis to which it is coupled by an idealized suspension system of specific characteristics. The wheel and chassis are assumed to be traveling along the road at 80 km/hr. As with the BI scale, the IRI scale is measured in terms of units of vertical movement of the wheel per unit length of road, and is normally quoted in meters per kilometer. Traditionally, the BI scale is normally quoted in millimeters per kilometer.

It seemed likely that to make a roughness-measuring machine of the desired performance, the design would have to be a variant of a static-profile measuring machine. In particular, it was believed that a device that could measure the spread of midchord deviations (described in the next section) would offer the most promise.

The problem therefore was twofold:

1. To examine, by computer simulation, the relationships between roughness and the spread of midchord deviations.
2. To design a machine that could reliably make the necessary measurements.

In practice, of course, the two were interrelated, with the simulation depending on the design of the machine and the design of the machine depending on the results of the simulation.

THE SIMULATION

Principle of Operation

The principle of the measurements is as follows. Two feet are rested on the road surface along the wheel track whose roughness is to be measured at a separation L (see Figure 1), and a probe is rested on the road surface midway between them. The vertical displacement is then measured between

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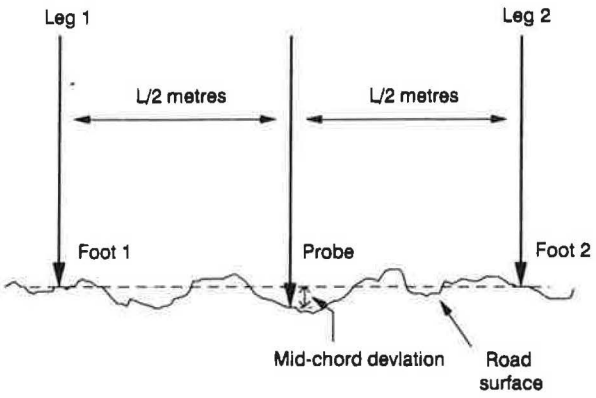

FIGURE 1 Measurement of midchord deviation.

the road surface under the probe and the center point of an imaginary line joining the two points where the road surface is in contact with the two feet. This displacement is the midchord deviation.

If measurements are taken at successive intervals along the road, then the rougher the surface, the greater the variability of the displacements. By plotting the displacements as a histogram, the variability can be estimated by measuring the range of, say, the central 90 percent of the data.

The concept of using the spread of midchord deviations as a means of assessing road roughness is not new. Two roughness indices have been proposed, QI, and MO, which are each based on the RMS values of two midchord deviations with different base lengths. They are described by Sayers et al. (*5*). The purpose of using two base lengths is to try and match the behavior of response-type road roughness measuring systems (RTRRMSs), which usually have two resonant frequencies, corresponding to the natural vibrations of the wheel and the chassis, respectively. The scales were suggested as standard numerics that could be calculated relatively easily from road profiles measured by rod and level.

However, (for simplicity) the proposed machine would use just one base length, (for speed of operation) it would take its own measurements without the need for rod and level, and (for ease of use) roughness would be derived with little calculation.

The International Road Roughness Experiment

In 1982, a major study, the International Road Roughness Experiment (IRRE), was carried out in Brasilia (*5*) to compare the performance of a number of different road roughness measuring machines and to calibrate their measures to a common scale. As part of this study, the machines were run over a series of test sections 320 m long, for four types of road surface—asphaltic concrete (AC), surface-treated, gravel, and earth.

One of the instruments used in the study was a TRRL Abay beam (*6*). This uses an aluminum beam (3 m in length, supported at each end by adjustable tripods that can be used for leveling. Running along the beam is a sliding carriage that has at its lower end a wheel of 250-mm diameter that is in contact with the road surface. A linear transducer inside the carriage measures the distance between the bottom of the wheel and the beam to the nearest millimeter and this was recorded at 100-mm intervals along the road. By successively relocating the beam along the length of the road section and repeatedly leveling the beam, the recordings provided a continuous sampling of the road profile.

Data from the Abay beam were available for 27 of the test wheel paths. Roughness on the IRI scale was computed from the beam road profile data while roughness on the BI scale was measured by a fifth-wheel bump integrator towed at 32 km/hr. Eight of the paths were on AC roads, five on surface-treated roads, seven on gravel surfaces, and seven on earth surfaces. Roughnesses ranged from 2.44 m/km on the IRI scale (1270 mm/km on the BI scale) for the best AC surface to 15.91 m/km (16 750 mm/km on the BI scale) for the worst earth surface.

Procedure

Given these road profiles, it was possible to carry out a computer simulation of performance. It is assumed that, for ease of operation, the machine mechanically amplifies the displacements by a factor of 10. If the rear foot is placed at a horizontal distance X from the start of the section, then the probe would be at a horizontal distance of $(X + L/2)$ from the start and the front foot at a distance of $(X + L)$. If the corresponding vertical distances at these points are Y_0, Y_1, and Y_2, then the value d, measured by the machine, is given by

$$d = 10 \times [Y_1 - 0.5 \times (Y_2 + Y_0)] \quad (1)$$

Taking measurements at successive positions along the road is simulated by using successively increasing values of X. The values of d are tabulated into different 5-mm ranges to create a histogram, and once 200 observations have been made, the range covered by the central 90 percent of the data points can be measured. This range is defined as D.

For each of the test sections, four simulation runs were carried out. In each run, a measurement was taken every 1.5 m, so that the observations covered virtually the entire test section. In the first run, the starting point was at the beginning of the test section. Subsequent runs started at 0.4, 0.8, and 1.2 m from the beginning. The main analyses are based on the mean of the four resulting values of D.

Choice of Base Length

In order to see what value of base length would produce the best estimate of roughness, the operation was simulated with values of L ranging from 0.6 to 3 m. Using the procedure described earlier, linear regressions were derived relating the value of roughness on the two measuring scales to D for different base lengths.

Figure 2 shows the R^2-values for these regressions. As can be seen, good correlations were found. On the IRI scale, the best correlations occur between 1.4 and 2.6 m. The highest value occurs at 1.8 m, so this was chosen as the standard base

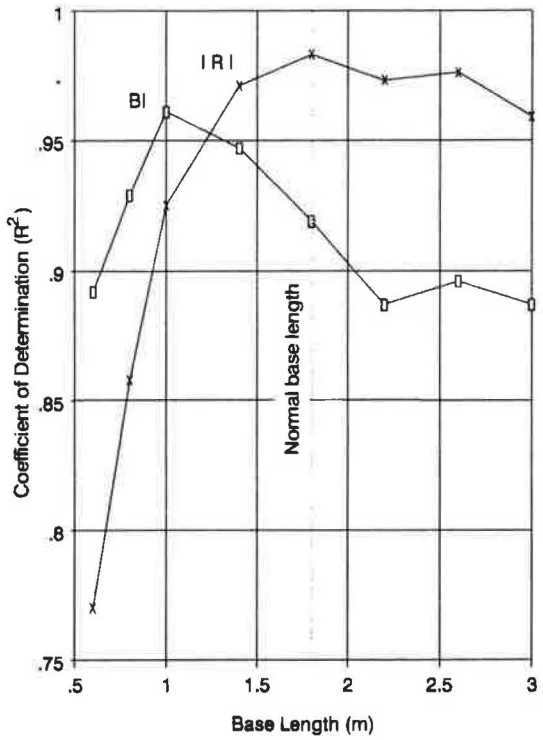

FIGURE 2 Roughness measuring accuracy for different base lengths.

length. Reducing the length below 1.4 m causes a sharp decrease in correlation.

Turning to the results for the BI scale, the answer is quite different. Here the best correlation is more sharply defined and occurs at a base length of 1 m. The degree of correlation is not as good as the best IRI value, but this is to some extent explained by the fact that the BI value was determined from an independent measurement of roughness and not from an analysis of the same profile data. The use of a 1-m base length is an attractive concept, because it would make a considerably more portable measuring device than the 1.8-m version. However, it would be a much poorer predictor of IRI.

The observed behavior in these simulations can be explained on the basis that the BI scale, which relates in these studies to a measuring speed of 32 km/hr, is more sensitive to short-wavelength surface undulations than the IRI scale, which relates to a measuring speed of 80 km/hr. At a length of 1.8 m, the device has a frequency response sufficiently similar to that of the IRI scale to make it correlate well with IRI. As the base length is reduced in length to 1 m, its frequency response becomes more similar to that of the BI scale.

Measurement of Data Spread

Tests were also carried out to find the best way of measuring data spread. In order to determine the central percentage of the data used to derive the value of D that would give the best answers, the performance over the test sections was again simulated. This time, the base length was fixed at 1.8 m and the roughness was measured on the IRI scale.

Linear regressions were again carried out between D and roughness, but on this occasion, the percentage of data points was varied. The larger the percentage, the larger the value of D. The resulting values of R^2 are shown in the following table. Of the values tested, 90 percent appeared to be the best choice, with smaller values and higher values giving distinctly poorer correlations.

Percentage of Data Points Used to Determine D	R^2
95	0.932
90	0.983
85	0.966
80	0.923

Simulation Results

A plot of roughness on the IRI scale against the mean value of D (derived from 90 percent of the data points) for each of the test sections is shown in Figure 3. Over the range of roughnesses examined, the points are a good fit to a linear regression passing close to, but not through, the origin. The coefficient of determination is over 0.98. Hence, it appears that a machine working on this principle can be used as a fairly accurate means of measuring roughness on the IRI scale. Table 1 presents the regression coefficients together with their standard errors.

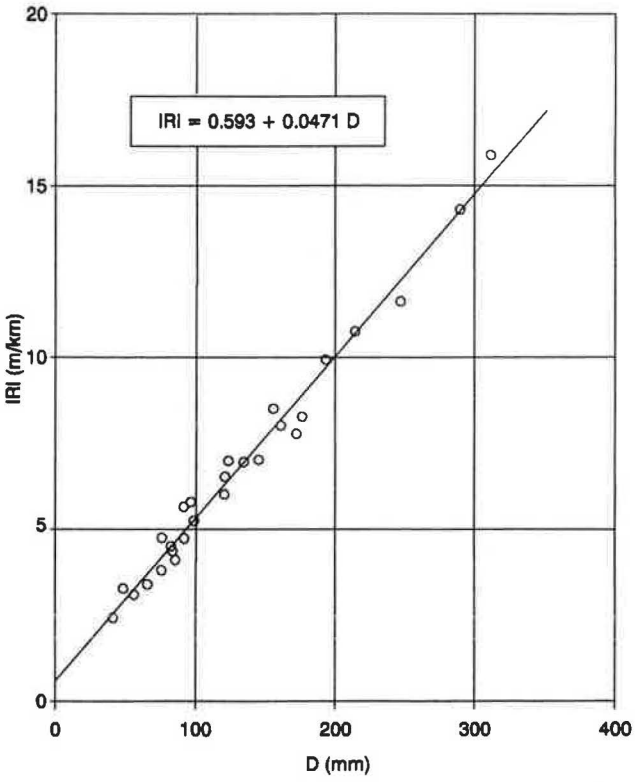

FIGURE 3 Relationship between IRI and D.

Figure 4 shows a plot equivalent to that shown in Figure 3, but this time measuring roughness on the BI scale. Once again, the points can be fitted to a linear regression passing close to the origin. However, the fit to the line is not as good as in the previous instance, and the coefficient of determination is reduced to just under 0.92. As mentioned earlier, this will be partly because of the fact that the BI value was derived from an independent measurement of roughness and not from an analysis of the same profile data.

The points shown in the figure distinguish between the different types of road surface, and on closer examination it can be seen that there are consistent differences between them. For example, all the results for gravel roads lie below the regression line. The analysis can therefore be improved by considering the different surface types separately, and the result of doing so is indicated in Table 1, which presents the regression coefficients. The coefficient of determination ranges from 0.914 on AC surfaces to 0.987 on surface-treated sections.

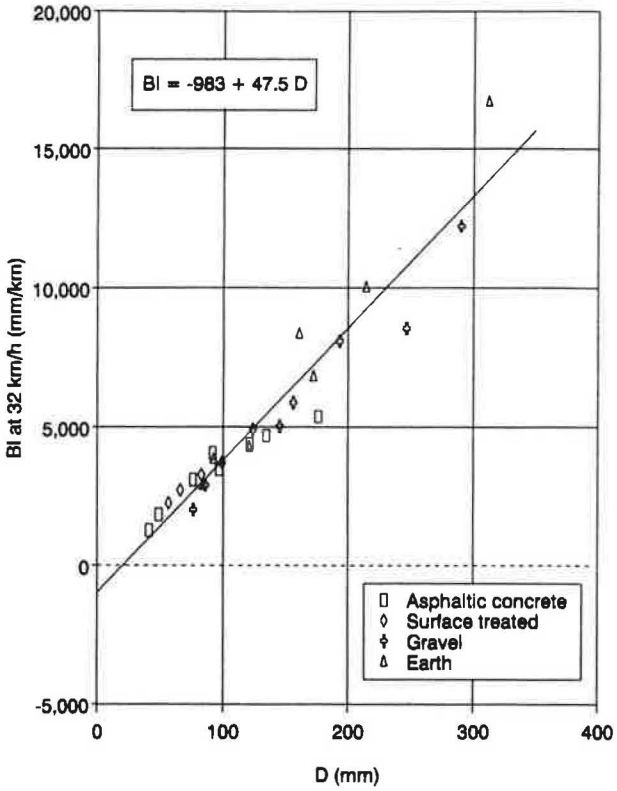

FIGURE 4 Relationship between BI and *D*.

THE MERLIN

General Description

The device that has been developed to take the measurements is called MERLIN (Machine for Evaluating Roughness using Low-cost INstrumentation).

TABLE 1 RESULTS OF REGRESSION ANALYSES (ROUGHNESS = $A_0 + A_1 \cdot D$)

Roughness scale	Surface type	A_0	A_1	R^2	N
IRI (m/km)	All	0.593 (0.185)	0.0471 (0.0012)	0.983	27
BI (mm/km)	All	-983 (423)	47.5 (2.8)	0.918	27
BI (mm/km)	AC	574 (401)	29.9 (3.7)	0.914	8
BI (mm/km)	ST	132 (220)	37.8 (2.5)	0.987	5
BI (mm/km)	GR	-1134 (676)	44.0 (3.6)	0.967	7
BI (mm/km)	EA	-2230 (797)	59.4 (4.4)	0.973	7

Notes: Bracketed values are one standard error
AC = Asphaltic concrete
ST = Surface treated
GR = Gravel
EA = Earth

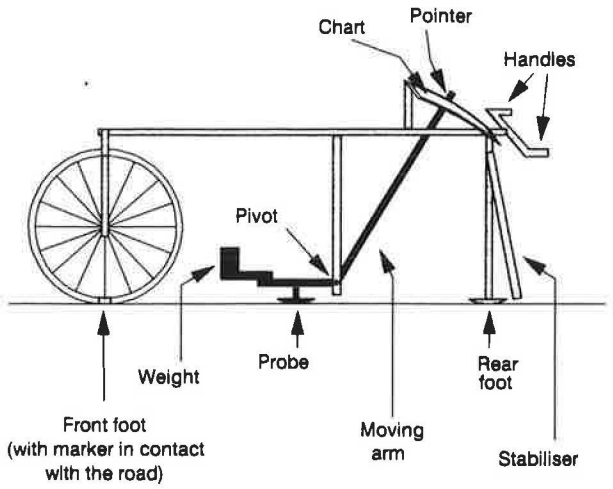

FIGURE 5 Sketch of the MERLIN.

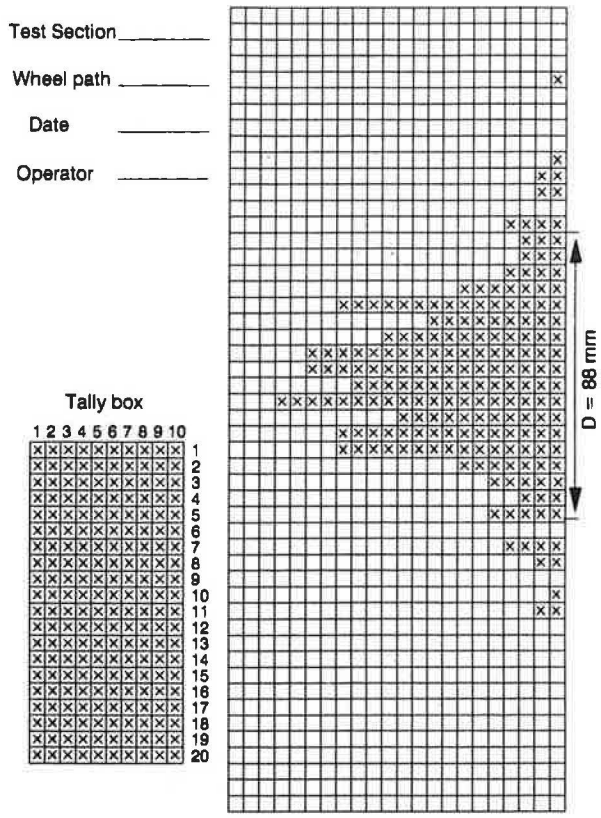

FIGURE 6 Specimen result.

Figure 5 shows a sketch of the device. For ease of operation, a wheel is used as the front leg; the rear leg is a rigid metal rod. On one side of the rear leg is a shorter stabilizing leg that prevents the device from falling over when taking a reading. Projecting behind the rear leg are two handles, so that the device looks in some ways like a long and slender wheelbarrow.

The probe is attached to a moving arm, pivoted close to the probe. The arm is weighted so that the probe moves downward, either until it reaches the road surface or the arm reaches the limit of its traverse. At the other end of the arm is attached a pointer that moves over the prepared data chart. The position of the parts is such that a movement of the probe of 1 mm will move the pointer by 1 cm. The chart consists of a series of columns, each 5 mm wide, and divided into boxes (see Figure 6).

Any variation in the radius of the wheel will result in a variation in the length of the front leg and this will give rise to unwanted movement of the probe. In order to overcome this unwanted movement, a mark is painted on the rim of the wheel and all measurements are taken with the mark at its closest proximity to the road. The wheel is then in its normal position.

Method of Use

In order to measure the roughness over a stretch of road, 200 observations are made at regular intervals. At each observation, the machine is rested on the road with the wheel in its normal position and the rear foot, probe, and stabilizer in contact with the road surface. The position of the pointer on the chart is then recorded with a cross in the appropriate column and, to keep a record of the total number of observations made, a cross is also recorded in the tally box on the chart.

The handles of the MERLIN are then raised so that only the wheel remains in contact with the road and it is moved forward to the next sample position, where the process is repeated. The spacing between the sample positions is not critical but readings must always be taken with the wheel in the normal position and so a spacing of one wheel circumference is the most convenient in practice.

When the 200 observations have been completed, the chart is removed from the MERLIN. The position midway between the 10th and 11th cross, counting the number of crosses in from one end of the distribution (e.g., from the top right in Figure 6), is marked on the chart below the columns. The procedure is repeated for the other end of the distribution. It may be necessary to interpolate between column boundaries, as shown by the lower mark of the example. The spacing between the two marks, D, is then measured in millimeters. This result is the roughness on the MERLIN scale. Road roughness, in terms of IRI or as measured by a towed fifth-wheel bump integrator, can then determined using one of the equations presented in Table 1.

Practical Details

Figures 7 and 8 show the MERLIN. For ease of manufacture, the main beam, the central and rear legs, the moving arm, the stabilizer, and the handles are all made from steel tubing of square cross section, 25 × 25 mm, with wall thickness of 1.5 mm. Joints are welded where possible, though the stabilizer and handles are fixed by bolts so that they can be removed for easier transportation. In order to strengthen the

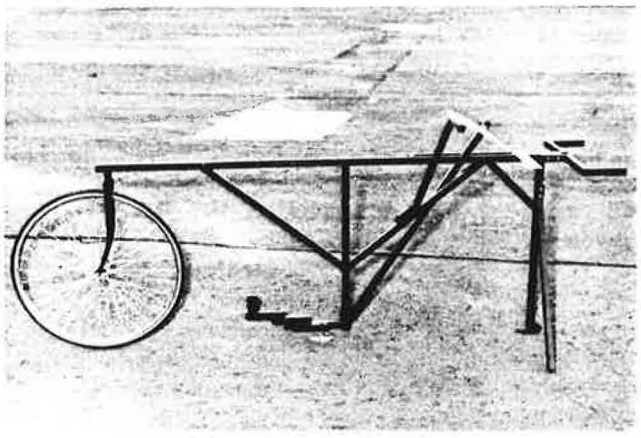

FIGURE 7 The MERLIN.

joints between the main beam and the legs, additional struts are used. The wheel is mounted in a pair of bicycle front forks and the tire has a fairly smooth tread pattern.

In order to reduce sensitivity to road surface microtexture, the probe and the rear foot are both 12 mm wide and rounded in the plane of the wheel track to a radius of 100 mm. The rounding also tends to keep the point of contact with the road in the same vertical line. The pivot is made from a bicycle wheel hub and the arm between the pivot and the weight is stepped to avoid grounding on rough roads.

The chart holder is made from metal sheet and is curved so that the chart is close to the pointer over its range of movement. In order to protect the arm from unwanted sideways movement, a guide is fixed to the side of the main beam, retaining the arm close to the beam. One end of this guide acts as a stop when the machine is raised by its handles.

The probe is attached to the moving arm by a threaded rod that allows both vertical and lateral adjustment. The position of the probe must be set so that the pointer is close to the middle of the chart when the probe displacement is zero, otherwise the histogram will not be central. Also, if the traverse of the probe does not pass centrally through the line joining the bottom of the tire and the rear foot, then it will be found that when the machine is tilted from side to side, the pointer moves. When correctly adjusted, leaning the machine over to one side so that the stabilizer rests on the road has little effect on the position of the pointer.

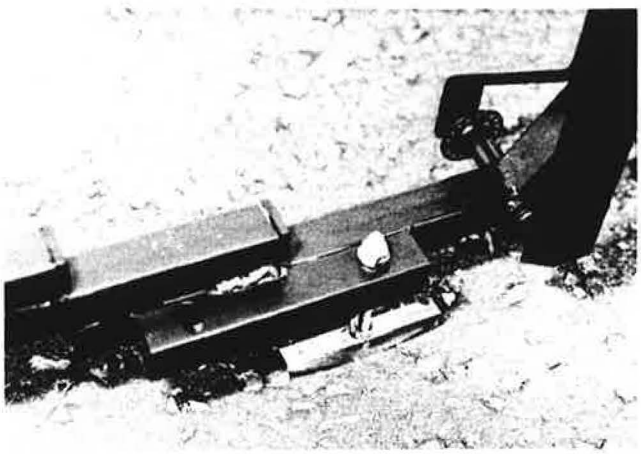

FIGURE 8 The probe and moving arm.

The only calibration required for the instrument is to check the mechanical amplification of the arm. This is normally done simply by measuring the movement of the pointer when a block of known thickness is placed under the probe.

When making measurements on a rough road, care has to be taken to ensure that fewer than 10 readings are at each limit of the arm's movement. If this is not the case, the probe can be moved to an alternative position that is twice as far from the pivot. This procedure reduces the mechanical amplification of the arm to 5 and halves the width of the distribution.

ACCURACY OF MEASUREMENT

As a simple check on the performance of the MERLIN on AC roads, the MERLIN and the Abay beam were used on a series of test sections of the TRRL test track. Roughness values on the MERLIN scale are shown plotted in Figure 9 against roughness on the BI scale as computed using the RMSD procedure with the Abay beam (*6*). The graph also shows the MERLIN-BI relationship for AC roads as presented in Table 1. Each point represents the mean of four MERLIN measurements. Although the check is by no means comprehensive, it does lend strong support to the calibration relationships derived from the simulation.

Whether using the MERLIN for calibrating other instruments or for direct measurement of roughness, two considerations about accuracy have to be borne in mind. The first consideration is that the MERLIN roughness for a road section is derived from a sample of observations and so is subject to a random sampling error. This error can be reduced by repeat observations on the same section. The second consideration is that there are systematic differences between the roughness scales that can only be reduced by repeat observations on different road sections.

Undulations in the road surfaces can be considered as surface waves with a spectrum of spatial frequencies. These spatial waves are converted into vertical oscillations of the wheel of a vehicle, the conversion factor depending on the vehicle's speed. The IRI, BI, and MERLIN scales and any RTRRMS being calibrated, all have different sensitivities to different spatial frequencies so they will correlate uniquely with each other only for surfaces with the same spectrum of spatial waves (spectral signature). In practice, surfaces will have different spectral signatures, though there are broad similarities, especially between the signatures of individual surface types. Hence, the relationship between the scales will not be unique and this gives rise to the systematic differences mentioned earlier.

If roughness is being measured directly on the MERLIN scale, then there are no systematic errors to contend with and the error falls with the reciprocal of the square root of the number of observations. A single measurement should have an RMS residual error of 8 percent, and taking the mean of four observations should reduce the error to 4 percent. When

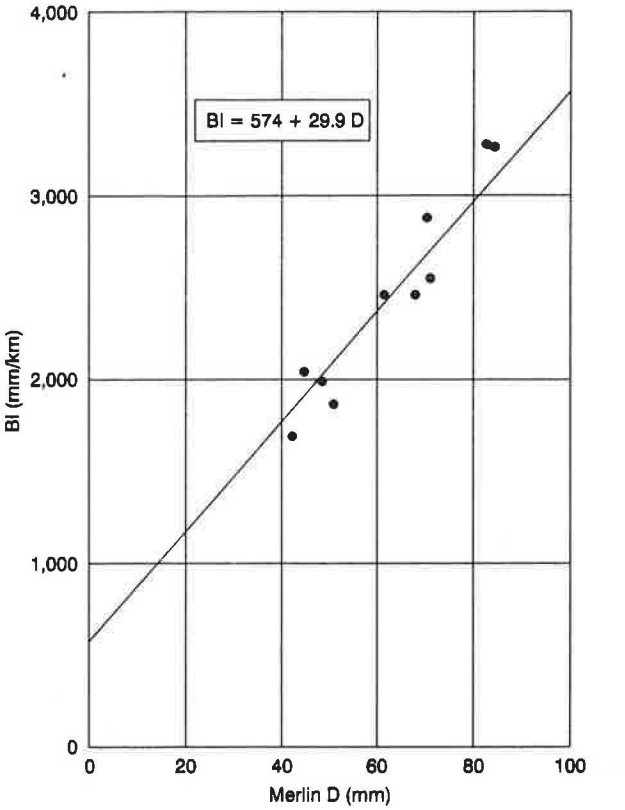

FIGURE 9 Calibration check on AC.

trying to measure roughness on the other scales, the systematic errors mean that repeat measurements do not produce as much improvement in accuracy.

Calibrating an RTRRMS on one of the standard roughness scales at a larger number of sites is better than making many repeat measurements at the same site. Moreover, particularly if working on the BI scale, these sites should have similar surfaces to those on which the RTRRMS is to be used. A number of other practical considerations should be borne in mind when measuring or calibrating and a useful guide is provided by Sayers et al. (7).

DISCUSSION OF RESULTS

The intention behind the MERLIN was that the device should be easy to use and reasonably accurate and yet able to be manufactured and maintained with the limited resources available in developing countries. Field experience so far has been satisfactory. A number of MERLINs have been made at TRRL and shipped overseas; other units have been made in developing countries from drawings provided by TRRL. To date, MERLINs have been used in 11 developing countries in South America, Africa, and Asia; in six of these countries the equipment was made locally at current prices of typically around U.S. $250.

One disadvantage of the device is that it is quite large and not easily transported within a vehicle. A shorter machine would be more convenient, but a reduction in the base length would lead to a poorer correlation with the IRI scale. Alternatively, a more portable design could be considered using a structure that dismantles. Although this is a possibility, it has been avoided because of the need to retain rigidity. Although the device is simple, it is able to measure displacements to better than a millimeter and this ability could easily be compromised by unwanted flexing of the structure.

Over the years, a number of road roughness scales have been proposed. Now the MERLIN scale, which correlates well with the IRI scale, can also be considered for applications such as identifying road maintenance intervention levels.

ACKNOWLEDGMENTS

This work forms part of the program of research of the Overseas Unit of TRRL. It is published by permission of the director of TRRL.

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Views expressed herein are not necessarily those of the Overseas Development Administration.

Results of Unsurfaced-Road Rating Surveys

ROBERT A. EATON AND SIDNEY GERARD

A method for rating unsurfaced roads has been developed, and a field manual has been prepared, to assist county, municipal, military, and township highway agencies in managing the maintenance of such roads. The rating method and strategies are compatible with the PAVER pavement management system developed by the U.S. Army Corps of Engineers and the American Public Works Association. An unsurfaced roads component of micro-PAVER is available, providing highway agencies with a more comprehensive roadway management system. This methodology has been implemented at a number of military installations throughout the continental United States and Alaska, as well as on the Dalton Highway in Alaska. These installations include Fort Riley, Kansas; Fort Ord, California; Fort Carson, Colorado; and Forts Wainwright and Greely, Alaska. The rating surveys and data gathered from these surveys have been analyzed.

About two-thirds of the highway systems in the United States and 90 percent of all roads worldwide are unsurfaced or lightly surfaced low-volume roads. No single, recognized management system is being used to effectively maintain these roads. The U.S. Army Corps of Engineers, the American Public Works Association, and others have developed pavement management systems (PMSs) for use on paved roads. Currently, these PMSs cannot be used for unsurfaced roads; however, a revised version of the U.S. Army Construction Engineering Research Laboratory's (CERL) Micro-PAVER that includes an unsurfaced-road component has been developed. An unsurfaced-road component that can stand alone or be used with any of these PMSs would give local highway agencies a comprehensive roadway management system more suitable for their needs.

The research effort to develop a method for rating and managing the maintenance of unsurfaced roads has been divided into three phases: Phase I, field manual development; Phase II, field validation and deduct-value model development; and Phase III, method implementation and development of PMS-software-compatible packages. Actual field survey results using this unsurfaced-road rating methodology have been obtained.

Phases I and II resulted in the publication of a field manual (*1*). The manual explains how to do a field inspection and calculate the unsurfaced road condition index (URCI), which is a measure of the road's overall condition and which corresponds to the pavement condition index (PCI) used in PAVER.

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The field inspections consist of windshield inspections and detailed measurements. Windshield inspections are performed by driving the full length of an unsurfaced road at 25 mph to determine the overall surface and drainage conditions four times a year (once each season). General estimates of maintenance needs and priorities can be made from this initial inspection. Measurements are the collection of detailed data on the roadway's surface and drainage conditions. After the initial inspection ride, a representative 100-ft-long section of road is selected for the actual measurements of distresses. The section should be permanently marked, so that future measurements will be taken in exactly the same location. In general, two sections per mile are enough.

The Phase I field manual identified six unsurfaced-road distresses and two drainage-related distresses, each with a separate index. As a result of the Phase II field validation (*2*), the two indices were combined. The manual currently lists the following seven distresses:

1. Improper cross section,
2. Inadequate roadside drainage,
3. Corrugations,
4. Dust,
5. Potholes,
6. Rutting, and
7. Loose aggregate.

For each distress, the severity and density are measured, and the deduct value is determined from graphs. The URCI can then be determined from all the deduct values.

Phase III was the development of a PMS-software-compatible package (*3*) and the implementation of the method at a number of locations throughout the United States. As previously mentioned, a PMS-software-compatible package based on the methodology in the Phase I manual has been developed by CERL and is available as micro-PAVER Version 2.1. The second objective of Phase III was the implementation of the method.

The method has been implemented at six locations. These sites are the Dalton Highway, Alaska (*4–6*); Fort Riley, Kansas (*7,8*); Ford Ord, California (*9*); Fort Carson, Colorado (*10,11*); Fort Wainwright, Alaska (*12,13*); and Fort Greely, Alaska (*14,15*). The individual site surveys provided the necessary data (distress measurements and deduct values) to establish unsurfaced-road data bases. These data were used to develop, assess, and modify maintenance practices. In addition, the data can be used as the basis for developing maintenance practices throughout the world.

TABLE 1 SAMPLE UNIT RATING FREQUENCIES FOR THE DALTON HIGHWAY, ALASKA

Sample unit ratings	No. of sample units 1986	1988	1989
Excellent	1	1	0
Very good	10	7	10
Good	10	13	10
Fair	0	0	1
Poor	0	0	0
Very poor	0	0	0
Failed	0	0	0
Total	21	21	21

TABLE 2 URCIs FOR THE DALTON HIGHWAY, ALASKA

Sample unit location (mi)	URCI 1986	1988	1989
16.5	86	69	73
30.0	80	82	74
61.0	72	55	58
79.0	67	68	78
88.0	74	75	69
134.0	65	75	77
149.0	65	73	53
203.5	76	65	78
216.0	65	69	70
229.5	60	72	61
255.5	72	65	55
266.0	74	62	67
281.0	68	57	73
301.0	66	61	68
310.0	85	87	78
313.0	78	62	76
334.0	72	71	56
355.5	75	66	59
370.0	69	66	64
383.5	68	57	59
395.5	61	70	70
Mean	71[1]	68.0[2]	67.4[2]
Std. dev.	7.2	8.0	8.2

[1] very good.
[2] good.

The survey data were analyzed manually as well as by the PAVER PMS. The individual site data were analyzed, and trends in the network ratings were related to maintenance allocations and practices. Finally, the ratings of the various site surveys were compared. Discussions on the results of the site survey data are presented in the following sections.

SITE SURVEYS

Dalton Highway, Alaska

The Dalton Highway, the only site with three years of data, was surveyed in 1986, 1988, and 1989. Table 1 presents a frequency distribution of sample unit ratings. These sample unit ratings are the URCIs calculated using the deduct values, which are based on the individual distress measurements. For the Dalton Highway sample units, ratings were recorded as excellent, very good, good, and fair. The data in Table 1 indicate that individual sample unit ratings have remained relatively constant or have deteriorated only minimally. Table 2 presents the individual URCI ratings for each sample unit, as well as the mean of each year's survey results. From these data, the ratings appear to have dropped slightly (71.2 for 1986, 68.0 for 1988, and 67.4 for 1989). Discussions with Alaska Department of Transportation personnel indicate that these slight drops in URCI may be attributable to reductions in maintenance funds and hence reduced maintenance actions. There is concern that if funding reductions continue, greater road degradation may result.

Fort Riley, Kansas

Fort Riley's unsurfaced-road network was rated in 1988 and 1989. The summaries presented in Table 3 indicate that during the 1988 survey all the poor, very poor, and failed sample units were tank trails, and all but one of the fair sample units

TABLE 3 SAMPLE UNIT RATING FREQUENCIES FOR FORT RILEY, KANSAS

	No. of sample units						No. of branches with average weighted URCI					
	All data		Tank trails		Roads and parking areas		All data		Tank trails		Roads and parking areas	
Rating	1988	1989	1988	1989	1988	1989	1988	1989	1988	1989	1988	1989
Excellent	48	41	21	26	27	15	5	1	1	0	1	4
Very good	108	114	49	58	59	56	20	22	6	7	14	15
Good	22	41	13	19	9	22	5	8	1	3	5	4
Fair	14	16	13	13	1	3	2	2	2	0	2	0
Poor	16	8	16	7	0	1	0	0	0	0	0	0
Very poor	3	1	3	0	0	1	0	0	0	0	0	0
Failed	1	0	1	0	0	0	0	0	0	0	0	0
Total	212	221	116	123	96	98	32	33	10	10	22	23

TABLE 4 DISTRESS MEASUREMENT SUMMARY FOR FORT RILEY, KANSAS

Branch	No. of sections	No. of sample units	Length of branch (mi)	Average weighted branch URCI 1988	1989
1STEF	2	3	0.95	67.7	75.3
AHAAR	1	2	0.40	79.5	78.0
AHASA	1	–[1]	–	–	–
CRSHA	3	7	4.34	78.2	54.8
DSTEF	1	1	0.12	67.0	78.0
EMPRC	3	3	0.95	80.3	84.4
GT11R	1	4	2.45	77.0	75.0
LMPRC	1	1	0.15	85.0	78.0
MPRC1	1	5	0.52	85.0	83.0
MPRC2	1	6	1.14	85.0	79.8
MPRC3	1	4	0.44	86.2	84.0
MPRC4	1	12	2.08	82.0	87.1
MPRCA	5	7	2.45	78.2	74.4
MPRCB	4	6	2.20	78.4	75.7
MPRCC	7	9	2.82	77.8	77.7
MPRCD	3	6	2.16	81.5	73.1
MPRCE	4	5	1.44	77.3	68.1
MPRCF	2	2	0.31	76.9	66.8
MPRCG	4	6	2.17	74.8	63.8
MPRCW	2	2	0.80	70.1	70.8
SRMAF	1	2	1.01	–	78.5
TTBLA	4	14	9.03	58.3	55.2
TTBRA	3	13	8.30	46.6	57.4
TTBUA	6	26	16.10	51.6	65.3
TTGRA	1	8	7.02	78.6	79.3
TTPUA	1	5	5.87	74.2	77.0
TTPUB	1	1	0.25	73.0	81.0
TTPUC	2	2	0.36	84.9	84.6
TTREA	5	28	20.31	77.7	81.4
TTYEA	3	23	19.34	78.6	80.8
TTYEB	1	3	2.20	86.0	81.7
WSEGR	1	2	0.30	68.5	66.5
WSSAR	1	1	0.07	65.0	54.0
WSTAR	1	2	0.60	74.5	68.5
Total	79	221	118.65	70.4[2]	72.8[2]
Sample unit std. dev.				10.4	9.0

[1] No data.
[2] Very good.

were also tank trails. The URCI data in Table 4 indicate that the tank trail branches (those branches with names that begin with TT) were in worse condition than the roadways and parking areas. The average weighted URCI was 68.5 for the tank trails and 78.1 for the roadways. On the basis of these results, the 1988 survey report recommended that repairs be made to the tank trails to accommodate the heavier traffic loads. Fort Riley road engineers accepted this recommendation, and the tank trails were repaired. The 1989 URCI ratings presented in Tables 3 and 4 indicate an improvement in the overall network from 70.4 to 72.8. The 1989 URCI for tank trails was 73.0, compared with 72.4 for roadways. This exemplifies the benefits a PMS can provide when used properly.

Fort Ord, California

The unsurfaced roads at Fort Ord were surveyed in 1988. Fort Ord is the only site that has been inspected just once. This road network was well maintained but needs to be reevaluated. Funding limitations have prevented a second survey. Table 5 presents the sample unit rating frequency distribution, with 20 of the 29 sample units inspected having very good ratings. Table 6 presents a summary of the branch ratings. The average weighted network URCI was 71.5.

TABLE 5 SAMPLE UNIT RATING FREQUENCIES FOR FORT ORD, CALIFORNIA

Rating	No. of sample units	No. of branches with average weighted URCI
Excellent	2	0
Very good	20	7
Good	4	2
Fair	3	1
Poor	0	0
Very poor	0	0
Failed	0	0
Total	29	10

TABLE 6 DISTRESS MEASUREMENT SUMMARY FOR FORT ORD, CALIFORNIA

Branch	No. of sections	No. of sample units	Length of branch (mi)	Average weighted branch URCI
AROWE	2	4	3.80	76.4
ARSKE	1	3	3.00	78.3
ARGDE	1	2	1.80	66.5
ARECE	1	2	1.50	75.0
ARCBE	3	5	2.60	74.2
ARWGE	1	2	1.40	81.5
ARPFE	3	4	1.30	78.9
AROCE	1	2	1.80	46.0
ARPF5	1	2	1.90	56.5
ARPCE	1	3	2.90	82.0
Total	15	29	22.00	71.5
Sample unit std. dev.				12.5

TABLE 7 SAMPLE UNIT RATING FREQUENCIES FOR FORT CARSON, COLORADO

Rating	No. of sample units 1987	1988
Excellent	4	6
Very good	18	38
Good	29	20
Fair	12	1
Poor	2	0
Very poor	0	0
Failed	0	0
Total	65	65

Fort Carson, Colorado

Fort Carson's unsurfaced roads were inspected in 1987 and 1988. The 1987 inspection indicated that road maintenance needed to be improved, because the network condition appeared to be in a downward trend. Significant reconstruction took place in 1988, and material was added to many of the branches. This repair substantially improved the overall road ratings. These improvements can be observed in summary results presented in Tables 7 and 8. Table 7 indicates that 12 sample units had fair ratings in 1987, as opposed to only 1 in 1988. In 1987, 29 units received good ratings, versus 20 in 1988. However, only 18 very good samples were taken in 1987, versus 38 in 1988. This data indicates the general improvement in sample unit ratings. Another indication of road condition improvement is presented in Table 8. The overall weighted network URCI was 67.6 in 1987 and 74.7 in 1988. These improvements, once again, are evidence of the benefits that can be reaped from using PMSs.

Fort Wainwright, Alaska

Fort Wainwright's unsurfaced-road network was inspected in 1988 and 1989. There is concern that road conditions at Fort Wainwright are deteriorating significantly. Tables 9 and 10 summarize the 1988 and 1989 road surveys. Road conditions significantly degraded between those two surveys. From Table 10, the average weighted URCI in 1988 was 77.6, versus 66.8 in 1989. In 1988, Fort Wainwright's unsurfaced-road network was one of the best, yet in 1989 this same network had the worst URCI of any survey presented here. Certainly this sig-

TABLE 8 DISTRESS MEASUREMENT SUMMARY FOR FORT CARSON, COLORADO

Branch	No. of sections	No. of sample units	Length of branch (mi)	Average weighted branch URCI 1987	1988
VR010	2	5	6.51	64.7	66.7
VR020	1	2	2.63	73.5	65.0
VR030	1	2	1.34	82.0	72.0
VR040	5	9	6.91	69.7	75.1
VR060	2	3	3.48	73.0	82.7
VR080	3	6	3.98	71.0	78.1
VR811	1	2	1.13	60.5	82.0
VR090	2	3	1.77	72.7	73.9
VR100	1	1	0.15	56.0	65.0
VR110	5	12	14.90	72.9	77.0
VR120	1	5	4.22	61.2	74.4
VR130	1	3	3.15	55.0	75.7
VR140	1	5	5.83	73.6	74.6
VR14A	1	2	1.90	61.5	74.5
VR150	1	3	3.00	54.3	68.7
VR15A	1	3	2.05	42.0	83.3
Total	29	65	62.95	67.6	74.7
Sample unit std. dev.				12.9	7.6

TABLE 9 SAMPLE UNIT RATING FREQUENCIES FOR FORT WAINWRIGHT, ALASKA

Rating	No. of sample units		No. of branches with average weighted URCI	
	1988	1989	1988	1989
Excellent	11	0	4	0
Very good	28	24	9	8
Good	6	16	3	6
Fair	5	8	1	3
Poor	0	2	0	0
Very poor	0	0	0	0
Failed	0	0	0	0
Total	50	50	17	17

nificant decline in ratings can be partially blamed on the extremely harsh winter of 1988 and 1989. However, the primary factor causing this degradation can be traced to budget and personnel cuts, which directly affect the amount and quality of maintenance. Efforts to restore some of these cuts are recommended.

Fort Greely, Alaska

Fort Greely's unsurfaced roads were also surveyed in 1988 and 1989. The results of these surveys are presented in Tables 11 and 12. The unsurfaced roads at Fort Greely are very well maintained. In fact, the 1988 rating was the highest of all the sites surveyed, and the 1989 rating was the highest for that year. However, the rating was 81.4 in 1988 and dropped to 76.5 in 1989. Facility maintenance personnel attributed this significant decline to budget and personnel cuts, which resulted in a reduction in road repairs. On the basis of these results, a 1990 survey is very important.

TABLE 10 DISTRESS MEASUREMENT SUMMARY FOR FORT WAINWRIGHT, ALASKA

Branch	No. of sections	No. of sample units	Length of branch (mi)	Average weighted branch URCI	
				1988	1989
PVES0	3	3	0.75	87.0	73.0
PRIV0	4	6	3.65	80.7	62.8
PSAG0	1	3	2.55	72.7	65.3
PCAN0	1	1	1.00	88.0	72.0
PTAN0	5	7	2.70	83.2	73.1
PSKI0	2	2	0.60	85.5	57.3
PFRO0	1	2	0.75	82.0	82.0
PAPP3	1	1	0.30	73.0	68.0
PASP0	3	4	2.00	91.7	78.7
PPER0	2	4	1.65	62.2	54.0
PBIR0	1	2	0.65	60.0	42.0
PBIL0	2	3	0.70	70.0	69.9
PGLA0	1	2	1.00	69.5	79.0
PALD0	1	3	1.40	71.0	42.3
POAK0	1	1	0.25	42.0	80.0
POLD0	1	2	1.10	80.0	67.5
PCHI0	1	4	1.40	77.8	77.3
Total	31	50	22.45	77.6	66.8
Sample unit std. dev.				12.4	13.7

TABLE 11 SAMPLE UNIT RATING FREQUENCIES FOR FORT GREELY, ALASKA

Rating	No. of sample units		No. of branches with average weighted URCI	
	1988	1989	1988	1989
Excellent	14	10	10	7
Very good	19	18	10	12
Good	2	7	1	2
Fair	0	0	0	0
Poor	0	0	0	0
Very poor	0	0	0	0
Failed	0	0	0	0
Total	35	35	21	21

SUMMARY

Table 13 presents a summary of the results of all the unsurfaced-road surveys conducted to date. The average weighted URCIs of each survey are presented as a function

TABLE 12 DISTRESS MEASUREMENT SUMMARY FOR FORT GREELY, ALASKA

Branch	No. of sections	No. of sample units	Length of branch (mi)	Average weighted branch URCI 1988	1989
PMEAO	1	6	7.40	75.5	66.0
PBOLO	1	2	1.65	72.5	68.0
PBEAO	1	4	4.10	86.3	80.0
PWESO	1	1	0.50	78.0	79.0
P3170	1	1	0.10	71.0	87.0
PPINO	1	1	0.20	93.0	81.0
PBUTO	1	2	0.70	85.0	82.5
PSIXO	1	1	0.25	93.0	98.0
PASTO	1	1	0.30	77.0	77.0
PCSTO	1	1	0.25	82.0	80.0
PDSTO	1	1	0.10	87.0	85.0
PEVEO	1	1	1.00	76.0	91.0
PDEHO	1	1	0.15	79.0	91.0
PFIRO	1	2	0.40	85.5	82.5
PEASO	1	1	1.00	92.0	91.0
PINCO	1	1	0.04	80.0	79.0
PSENO	1	1	0.45	83.0	81.0
PSHAO	1	1	0.30	86.0	84.0
P6320	1	1	0.05	65.0	74.0
P33MO	1	3	1.15	87.7	82.7
PLANO	1	2	1.35	95.0	90.0
Total	21	25	21.44	81.4[1]	76.5[1]
Sample unit std. dev.				10.0	9.5

[1]Very good.

of site and year of the survey. The 12 surveys produced average URCIs ranging from 66.8 (good) to 81.4 (very good). The rating of the individual sites appeared to be comparable. When proper maintenance was available, the URCI showed improvement; conversely, when the URCI declined, further study found that maintenance funds and personnel had been cut. The Alaska sites should be carefully monitored to further assess the effect of budget and personnel cuts.

ACKNOWLEDGMENTS

The authors especially thank K. Leland and L. Stringer of the U.S. Army Engineer District, Tulsa, for their assistance in data collection and field support. In addition, the authors express their appreciation to W. Allen and S. Ketcham of the U.S. Army Cold Regions Research and Engineering Laboratory (CRREL) for their technical reviews of the paper, and D. Cate of CRREL for his editorial review. Funding was provided by the U.S. Army Facilities Engineering Applications Program, Ft. Belvoir, Virginia.

TABLE 13 NETWORK AVERAGE-WEIGHTED URCI COMPARISONS

Location	1986	1987	1988	1989
Dalton Highway, AK	71.2	–[1]	68.0	67.4
Ft. Riley, KS	–	–	70.4	72.8
Ft. Ord, CA	–	–	71.5	–
Ft. Carson, CO	–	67.6	74.7	–
Ft. Wainwright, AK	–	–	77.6	66.8
Ft. Greely, AK	–	–	81.4	76.5

[1] No data.

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Evaluation and Rating of Gravel Roads

DONALD M. WALKER

Roadway maintenance management systems and pavement management systems are growing in popularity with road maintenance agencies. These systems require an objective evaluation of roadway pavement conditions, including a numerical rating scale to be used in analysis and priority selection procedures. An evaluation and rating system for gravel roads has been developed. The evaluation and rating give primary consideration to drainage, crown, and adequacy of the gravel thickness. Other measures of distress, such as washboarding, dust, ruts, and potholes, are also considered; however, they are considered as secondary indicators of roadway conditions. The procedure highlights an understanding of typical distress along with the causes and remedies for the individual types of distress. The rating procedure is closely linked to the appropriate maintenance or rehabilitation treatment.

Roadway maintenance management and pavement management systems are growing in popularity with road maintenance agencies. These systems require an objective evaluation of roadway pavement conditions. Normally, they require a numerical rating scale to be used in the analysis and priority selection of projects. Although considerable research and development effort has been devoted toward evaluation and rating of asphalt and concrete pavements, only a limited amount of work has been completed on rating systems for gravel surface roadways. The U.S. Army Corps of Engineers has developed one system (*1*).

The Transportation Information Center at the University of Wisconsin in Madison has developed a visual rating evaluation system for asphalt, concrete, and gravel roads, called the Gravel-PASER system (*2*). The system is in use by city, county, and town governments in Wisconsin and is being incorporated into various pavement management systems. It is being used on road systems with urban arterials, county highways, and low-volume town and county roads. Actual use and training of field personnel is best done with the Gravel-PASER Manual, which incorporates many photographs to illustrate the rating procedure. Individual maintenance agencies may want to develop their own version of a rating and evaluation procedure that incorporates local conditions.

PAVEMENT MANAGEMENT SYSTEMS

The development and use of a pavement management system has provided many benefits to agencies with road maintenance responsibility. Such a system is an organized approach to make the most effective use of limited budgets. By documenting the actual conditions of roads, realistic budgets can be developed and timely repairs can be scheduled. The development of an overall plan for the roadway systems helps agencies develop a meaningful budget and plan for future needs. The detailed information provided by a pavement management system is also effective in gaining public support for an adequate budget.

Key steps in developing a roadway management system should be taken. As a minimum, the roadway system must be broken into individual roadway segments of a similar pavement thickness and traffic volume. Inventory information on the segment, such as geometrics, traffic volume, and functional classification, is normally included. Also, some assessment of the roadway condition must be provided. Generally, through detailing the type of distress, its extent, and its severity, an overall indicator of condition is developed.

The management system can then develop cost and recommended maintenance rehabilitation strategies on the basis of condition information. A system can further order projects by priority in analysis to maximize cost benefits. Many systems have completely automated the process. Others simply include an inventory and condition survey and require the user to develop priorities and cost estimates.

Agencies with many miles of low-volume roads may not feel a sophisticated pavement management system is justified. However, a basic inventory and condition rating can be developed using local personnel with limited training. Because most agencies routinely review road conditions in the development of their budgets, little additional work is required to document these conditions. An agency can begin with a simple system that can evolve into a more sophisticated pavement management system as the benefits are demonstrated.

ROAD CONDITION EVALUATION AND RATING

All pavement management systems require the evaluation of pavement conditions. Even without a formal pavement management system, a basic record of pavement conditions is useful to maintenance supervisory staff. However, the rating and evaluation system must reflect the needs of the agency. With computerized systems, users are tempted to collect large amounts of data. For obvious cost reasons, only data that will be used should be collected. A simple approach that produces results is more likely to continue in use than a large, complicated system with limited benefits.

The pavement rating scale used in the PASER manuals is a visual rating. The scale is based on the type and severity of common defects. The overall rating scale is directly related to the type of maintenance or rehabilitation most appropriate for that roadway segment.

The PASER pavement rating scale requires the use of judgment by the person doing the rating. In a simple system, the best time to make those evaluations and judgments is when

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the rating team in the field, looking at the roadway. This rating system has the benefit of requiring the rating team to look for critical defects and answer the difficult questions on location. Having supervisory personnel inspect for critical information in the field, make a judgment on the overall road condition, and recommend maintenance or rehabilitation has its benefits. If that decision cannot be reached by a visual inspection, the staff is motivated to recommend additional testing, sampling, or other evaluation. The decision is not delayed and does not become an arbitrary analysis of numerical values.

Inspection and rating can be done by technical staff, managers, or elected officials. This system is designed to be simple and effective. Experience indicates that elected officials with a limited technical background can easily be trained to do the rating. The system works best when rating is done by both management and staff. This approach improves communication and significantly improves implementation of maintenance or reconstruction recommendations.

RATING GRAVEL ROADS

Rating and evaluating gravel-surfaced roads differs from rating paved surfaces. Gravel road surface conditions change quickly. Heavy rains, heavy local traffic, or recent maintenance activities can significantly change many of the gravel road surface characteristics. Therefore, gravel road rating should be based primarily on three major factors.

Because the purpose of the rating and evaluation is to determine a need for future maintenance and rehabilitation, the rating system should reflect the major factors that affect the performance of the roadway. These factors are roadway crown, drainage, and adequacy of the gravel layer. Performance of a gravel road under traffic depends heavily on these factors.

The road crown and drainage system, primary factors in the evaluation, can be readily observed. Determining the adequacy of the gravel layer may be more difficult. Whether the thickness is sufficient and the aggregate quality is acceptable to carry the traffic using the road should be determined. Actual sampling of material thickness and quality would be helpful. However, performance of the roadway under existing traffic can provide a good indication of the adequacy of the gravel layer. Load-related distress, such as rutting, and failure, such as potholes, are obvious indicators of inadequate load-carrying abilities.

Other surface distresses will be of secondary interest. Washboarding, loose rock, and dust are primarily indications of traffic distress and the adequacy of recent maintenance activities. Although these factors are important in planning routine maintenance, they are less critical for planning major rehabilitation or reconstruction.

ROAD CONDITIONS AND DEFECTS

A gravel road is best given an overall rating through observation of individual defects. These defects can be combined to provide the information necessary to make an overall assessment of road conditions. This system will consider three primary conditions: crown, drainage, and gravel layer, along with the secondary effects of surface deformation and defects.

Crown

A gravel-surfaced road must be built so that water drains quickly off the roadway. Thus, a crown is built into the road. Normally, a gravel road should have between ½ and ¾ in. per ft of width of fall (crown) from the center of the edge of the roadway. No ponding or depressions that will collect water should be in the road. The shoulder or edge of the roadway must transition smoothly into the ditch. High shoulders or secondary ditches trap water and soften the roadway. A carpenter's level mounted on a straight 2 × 4 may be useful in determining the exact amount of crown. However, an adequate crown and the absence of features such as ruts and secondary ditches can easily be observed.

Drainage

The drainage system adjacent to the roadway must be adequate to handle surface water flow. The system includes primarily ditches and culverts. The ditch must be wide and deep enough to accommodate all surface water and have an adequate slope so that water does not pond or cause erosion. Generally, a V-shaped or rounded ditch is provided. Having the bottom of the ditch a minimum of 1 ft below subgrade to provide adequate drainage is desirable. Evidence of serious ponding, flooding, and erosion can be seen at almost any time of the year. A review of slope and ditch adequacy may be more easily made during wet weather conditions. Detailed surveys or wet weather inspection would be useful on individual projects planned for grading or reconstruction.

Roadway culverts and bridges are also important elements to be reviewed. Collapsed culverts or silt- and debris-filled culverts or bridges are indications of poor drainage. Adequate headwalls and culvert apron endwalls help minimize erosion.

Rating drainage on a roadway segment requires an assessment of the overall condition and identification of spot problems. Localized conditions are commonly found to need cleaning or repair. Making an overall assessment, such as the percentage of roadway that needs ditch cleaning versus major ditch and culvert construction, is important. Those assessments will help determine the extent of budgeted maintenance or rehabilitation required.

Adequate Gravel Layer

The third major factor to consider is the adequacy of the gravel layer. The gravel pavement thickness must obviously be designed to accommodate the traffic loads and soil conditions. Therefore, no simple and uniform guidelines exist. In evaluating and rating this characteristic, signs of distress related to inadequate pavement strength should be sought. Failures from heavy loads take the form of rutting and potholes. Minor surface rutting (less than 1 in.) can occur from traffic dislodgement of gravel. Deeper rutting (over 1 in.) is a better indicator of actual strength limitations related to the gravel layer. Isolated potholes may indicate isolated conditions. More extensive potholes and breakdown of the surface are indications that an adequate layer does not exist.

Understanding the maintenance record of a road also improves the ability to rate and evaluate conditions. If frequent regrading is necessary to prevent rutting and repair potholes, an adequate gravel layer may not exist. Obviously, roadway strength is related to drainage and subgrade support, as well as gravel thickness. A gravel layer that would normally be adequate may not perform well if the roadway is frequently flooded or in an area of a very high water table.

If surface distress, such as rutting and potholing, is not sufficient to evaluate the adequacy of a gravel layer with confidence, more field investigation is recommended. Several test holes can provide information on the thickness of the gravel layer. A visual inspection of the aggregate may indicate poor gradation. Laboratory testing of aggregate properties is even more useful.

SURFACE DISTRESS

The following defects are important to consider when developing an overall surface rating. Records on the extent and severity of these types of defects, when monitored from year to year, can show how well roadways are performing. The rate of change and development of surface defects can be helpful in selecting between routine maintenance and major rehabilitation.

Washboarding

Washboarding (corrugations) of an aggregate surface is a common distress under traffic loading. Washboarding provides an uncomfortable ride and can be a safety hazard. Slight to moderate (1–3 in.) washboarding can normally be corrected by routine grading. Heavy washboarding may be an indication of the need for additional gravel.

Potholes

Potholes may develop as an isolated defect. These require spot-patching or maintenance from a safety standpoint. Extensive (over 25 percent of the area) and deep (over 4 in.) potholes are an indication of lack of strength and the need for more major rehabilitation and the addition of gravel. Potholes trap water and can speed surface deterioration if routine maintenance is not provided.

Rutting

Rutting is another important defect to consider. Minor (less than 1 in.) rutting in the wheel path may be simply an indication of a heavy traffic volume. Routine regrading and maintaining good surface drainage can remedy this defect. Deeper rutting (over 3 in.) may indicate lack of gravel thickness or subgrade support. This defect is very serious and usually indicates that major reconstruction is required.

Dust

Dust from traffic is also a common occurrence on a gravel road. The gradation of the gravel, weather conditions, and traffic volumes will determine the extent and severity of dust. Since heavy dust conditions remove necessary fines from the roadway, this defect can be an indicator of future maintenance problems. Thick dust that obscures traffic can create obvious safety problems. A dust palliative is useful, especially near populated areas.

Loose Aggregate

Dusty conditions and the resulting loss of fine aggregate can produce an excess amount of loose large aggregate on the gravel surface. Under traffic, this loose aggregate can tend to collect between wheel paths and along the side of the road, creating a driving hazard and affecting drainage. Minor amounts of loose aggregate can often be remixed by routine grading. Large accumulations (over 4 in.) of loose aggregate can impede drainage and indicate a loss of the strength of the remaining gravel layer.

GRAVEL RATING SCALE

A simplified 5-point rating scale has been developed. Each category is intended to indicate conditions directly related to the need for maintenance or rehabilitation. The ratings may be thought of as follows:

5 (Excellent): A newly constructed road. Excellent crown, drainage, and gravel layer.

4 (Good): Recently regraded with good crown and drainage and adequate gravel layer.

3 (Fair): Needs routine regrading or minor ditch maintenance.

2 (Poor): Needs additional aggregate or major drainage maintenance.

1 (Failed): Complete rebuilding required.

The rating scale is discrete, and other ratings (2.5, for example) are not encouraged.

Table 1 contains a description of the individual ratings with the typical distress and recommended maintenance or rehabilitation procedures. Roadways will not have all types of distress at any particular time. They may only have one or two of the individual distresses.

IMPLEMENTATION

Establishing the limits of individual roadway segments requires initial planning. Elements of a segment should all have similar pavement thickness, traffic volume, and function. A segment should be limited by what would be reasonable for individual maintenance or reconstruction projects. Smaller segments can be created to isolate different conditions. Typical gravel road segments in Wisconsin seem to average 1 mi or slightly more.

TABLE 1 RATING SYSTEM

Surface rating	Visible Distress*	General condition/ Treatment measures
5 Excellent	No Distress. Dust controlled. Excellent surface condition and ride.	New construction - or total reconstruction. Excellent drainage. Little or no maintenance needed.
4 Good	Dust under dry conditions. Moderate loose aggregate. Slight washboarding.	Recently regraded. Good crown and drainage throughout. Adequate gravel for traffic. Routine maintenance may be needed.
3 Fair	Good crown (3" - 6"). Ditches present on more than 50% of roadway. Gravel layer is mostly adequate, but additional aggregate may be needed at a few locations to help correct washboarding or isolated potholes and ruts. Some culvert cleaning needed. Moderate washboarding (1" - 2"), over 10% - 25% of the area. Moderate dust, partial obstruction of vision. None or slight rutting (less than 1" deep). An occasional small pothole (less than 2" deep). Some loose aggregate (2" deep).	Shows traffic effects. Regrading (reworking) necessary to maintain. Needs some ditch improvement and culvert maintenance. Some areas may need additional gravel. *Note: Individual roadways may not have all of the types of distress listed for any particular rating. They may have one or two types.
2 Poor	Little or no roadway crown (less than 3"). Adequate ditches on less than 50% of roadway. Portions of the ditches may be filled, overgrown and/or show erosion. Some areas (25%) with little or no aggregate. Culverts partially full of debris. Moderate to severe washboarding (over 3" deep) over 25% of area. Moderate rutting (1" - 3"), over 10% - 25% of area. Moderate potholes (2" - 4"), over 10% - 25% of area. Severe loose aggregate (over 4").	Travel at slow speeds (less than 25 mph) is required. Needs additional new aggregate. Major ditch construction and culvert maintenance also required.

TABLE 1 (*continued on next page*)

TABLE 1 (*continued*)

Surface rating	Visible Distress*	General condition/ Treatment measures
1 Failed	No roadway crown or road is bowl shaped with extensive ponding.	Travel is difficult and road may be closed at times.
	Little if any ditching.	Needs complete rebuilding and/or new culverts.
	Filled or damaged culverts.	
	Severe rutting (over 3" deep), over 25% of area.	
	Severe potholes (over 4" deep), over 25% of area.	*Note: Individual roadways may not have all of the types of distress listed for any particular rating. They may have one or two types.
	Many areas (over 25%) with little or no aggregate.	

Inventory Date_____
By_____

SEGMENT & LOCATION
Road/Name_____ Segment No._____
From_____ To_____
Length_____

USE & CLASSIFICATION
Road Function_____ Avg. Daily Traffic_____
Land Use_____

ROADWAY CONDITION DATA
Crown_____ Gravel Depth/Quality_____
Ditch and Culvert Adequacy_____
Roadway Condition Rating_____
Special or Spot Problems_____
Comments_____

GEOMETRICS
Width of Traveled Way_____
Horizontal Alignment Rating_____
Vertical Alignment Rating_____
R/W Width_____
Comments_____

OTHER
Comments_____

IMPROVEMENT HISTORY

Year	Work Completed	Estimated Cost

FIGURE 1 Sample inventory form.

Rating	Action Required	Miles
5	None	9.3
4	Minor Grading	20.7
3	Minor Ditching/Reshaping	58.5
2	Added Gravel/Drainage Rehab	39.8
1	Reconstruction	19.2
	Total Miles of Gravel Roads	147.5

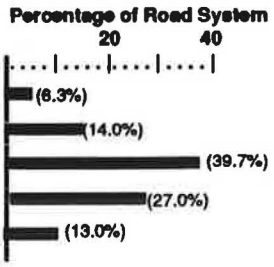

FIGURE 2 Surface condition ratings.

Experience indicates that the field rating process can easily cover 20 to 40 mi per day. Small agency networks have been completed in one day, but larger agencies may require more. The survey should be done annually, at the same time of year. Fall and Spring seem to be convenient and useful for budget development.

Rating an individual roadway segment usually involves evaluating conditions over a considerable length (1 mi or more). Because no roadway segment will be entirely consistent, averaging of conditions is necessary. Small or isolated conditions should not influence the overall rating, but should be noted for maintenance. The overall purpose of the condition rating is to provide a relative comparison between pavement segments. The rating system should be used to keep the conditions in relative order. That is, those rated 3 should all be better than those rated 2 and not as good as those rated 4. Within a specific rating, of course, not all roadways will be identical.

A sample inventory form is shown in Figure 1. Collecting past maintenance and construction information on the inventory form is useful. This information can be used in selecting future maintenance. Information on localized problems (which only occur once) is also useful for maintenance scheduling but not critical for decisions on major roadway reconstruction. Individual agencies are strongly urged to develop their own inventory form to fit their needs and the complexity of their system.

APPLICATIONS

This simple procedure allows decision makers to compare the conditions of road segments. Documenting poor road conditions helps to assign funds to the roads most in need of work. Listing roadway improvements by category has also been found helpful. That categorization is a simple listing of all roads that need routine minor regrading and ditch maintenance (rated 3), a separate listing of those that need additional gravel and major ditch cleaning (rated 2), and a listing of those needing complete reconstruction (rated 1). A review of these lists is helpful in selecting projects that may not appear on a priority listing of the worst roads. Because a priority listing is usually oriented toward worse conditions, ditch cleaning and routine maintenance projects may not surface until the road segment is completely deteriorated. Therefore, a review of individual listings by category helps balance decision making for cost-effective budgets.

One of the most important benefits of rating pavements is that decision makers are given an understanding of the overall road conditions. Figure 2 shows graphically the condition of a roadway system. If a road system is only in fair or poor condition, this type of data display can be very effective in convincing decision makers that additional improvements are necessary. The display can also indicate the benefits received from previous budget allocations. Watching this representation of road conditions from year to year is helpful in assessing the effectiveness of budgeted road funds. Once the rating has been completed, this information is easy to assemble and can be a significant benefit.

SUMMARY

A simplified gravel road rating procedure has been developed to assist agencies in implementing pavement management systems. Local agencies with low-volume roads can use the procedure as part of a comprehensive pavement management system or as a simple maintenance planning and budgeting tool.

The rating system, which uses a visual inspection approach and is linked directly to required maintenance and rehabilitation, uses information normally collected and understood by local maintenance supervisors and elected officials. Experience has shown that this rating procedure helps local officials develop budgets based on need. Better decision making and adequate budget preparation are the benefits of gravel road evaluation and rating.

ACKNOWLEDGMENTS

Financial support for development of this evaluation system was provided by FHWA, the Wisconsin Department of Transportation, and the University of Wisconsin—Extension.

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The views and findings expressed in this paper are those of the author and do not necessarily reflect those of the funding agencies.

Observed Behavior of Bituminous-Surfaced Low-Volume Laterite Pavements

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The deteriorating condition of paved-road networks, the need to upgrade gravel roads to bituminous standards, and the limited resources available are challenging highway administrators and managers in developing countries, as well as donor agencies, to find less costly solutions to establish efficient and integrated transport systems. Information on the behavior and performance of bituminous-surfaced road pavements with a base built from as-dug laterite has been obtained from several countries. Most of these roads, carrying a wide range of traffic from the lower end of low-volume roads to about 2,400 veh/day/lane, have performed well for 10 or more years; in many cases, their useful life can be extended by simple resealings. Data analysis included long-term variation of pavement condition (e.g., roughness and cracking) and deflection, influence of load and temperature on deflection, correlation between Benkelman beam and Dynaflect deflections, lateritic gravel characteristics, and strength parameters. The conclusion is that laterite bases can perform as well as crushed-stone or stabilized laterites under a wide range of circumstances, at a cost of about 20 to 30 percent of these more expensive materials. The key factors in the performance of laterite bases appear to be a high degree of compaction, well-drained subgrade, and rigorous material selection in the borrow pits for application in the road. Further research is needed for more precise definition of the relationships between laterite characteristics, traffic loading, environment, and pavement performance.

The deteriorating condition of paved-road networks, the need to upgrade gravel roads to bituminous standards, and the limited resources available have led highway administrators and managers in developing countries to face a stark dilemma. Should they continue to adopt restrictive imported specifications that result in higher construction and rehabilitation costs or accept a more widespread use of lower-cost local materials and risk the possibility of increased future maintenance costs?

The observed behavior and performance of pavements with a base built with as-dug laterites indicate that this material can be used under a wide range of environmental and traffic conditions to build and rehabilitate roads. A minimum total road transportation cost resulting from this practice would make it easier for developing countries to establish an efficient and integrated transport system. Laterite bases can perform as well as crushed stone or stabilized laterites, and marginally better in some cases, under a wide range of circumstances. Their cost is about 20 to 30 percent of these more expensive materials, and their use will not incur extra road maintenance or vehicle operating costs.

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Laterites are soils with a vesicular structure. Their colors range from yellow to red, sometimes with dark shades, frequently resembling a slag. Lateritic soils clay fraction (i.e., fraction passing 2 micra) shows a molecular silica/sesquioxide ratio (or SiO_2/R_2O_3) of less than 2 and low expansibility. Laterites occur in tropical and subtropical areas of Australia, South and Central America, Africa, and Asia and have long been used in the construction of roads, airports, and buildings.

Laterites are tropical soils that have been produced by advanced weathering accompanied by a relative enrichment in iron and aluminum sesquioxides (Fe_2O_3 and Al_2O_3, respectively) because of the decomposition of primary minerals and the removal of bases and silica, as described by Netterberg (*1*). Autret (*2*) has made a distinction between (a) lateritic fine soils or laterites; (b) lateritic gravels or granular laterites, used in road technology for base and subbase construction of paved roads, surfacing of unpaved roads, and in some cases as aggregate for surface treatments or asphalt concrete surfacings; and (c) cap rock (cuirasse lateritique), which are indurated concretions resembling slags, broken down by bulldozer blade or ripper. The laterites considered in this paper approach the second classification (b) above and consist mainly of gravel-sized concretions.

In general, laterites are among the more difficult materials to locate, and local prospecting may be required in many cases for precise location. However, the use of aerial photography in the search for laterites has been well documented (*3,4*). In the field, useful vegetation indicators can be developed on a regional basis, because laterites tend to be infertile. Charman (*5*) provides a good summary of guidelines for finding suitable sources of lateritic concretionary materials.

NATURAL VERSUS PROCESSED MATERIALS

The use of untreated local materials, such as laterites, has a considerable potential for cost savings in pavement construction and rehabilitation. However, when these materials do not satisfy the normal requirements for untreated road pavement materials, the alternative of improving locally occurring materials with stabilizing agents such as cement, lime, and bitumen has frequently been adopted (*6–9*). One other alternative is to improve the inadequate material through mechanical stabilization with sand or crushed rock. Two successful examples of the latter are the Tahoua-Arlit road in Niger, where some sections had the base course built with a 50–50

percent mixture of substandard laterite and crushed limestone (*10,11*), and the 40-km section of the Ouagadougou-Koudougou road in Burkina Faso (*12*).

Notwithstanding well-documented successes of the alternatives mentioned above, the additional cost (including relatively high proportions of foreign exchange) of providing the stabilizing agent, processing mixtures, or crushing and hauling rock has to be met. Moreover, despite the extra costs incurred to stabilize the otherwise substandard material, in many field examples the performance of the resulting pavement has not been demonstrably better than pavements built with the as-dug materials. The following are a few examples.

1. The Belem-Brasilia road in Brazil was paved from 1974 to 1976. The laterite-cement pavement showed premature failures, whereas as-dug laterite bases performed well. Although the reason for this unexpected soil behavior is not clear, several potential problems related to cement or lime stabilization of laterites have been cited by Netterberg (*1*): alkali-silica, carbonate, and alumina reactions; organic matter inhibition of stabilization; and unusual reactions with cement and lime (probably because of extremely high or low pH values). Carbonation has been shown to inhibit the formation of cementitious products in soil-lime and soil-cement reactions, thus adversely affecting pavement performance (*13*). Methods of predicting some of these problems have been described (*14,15*).

2. The South Bank Road in The Gambia was constructed from 1964 to 1968. The Brikama–Ba-Kalagi section was constructed with as-dug laterite. The Serekunda-Brikama and Kalagi-Soma sections were constructed with cement-stabilized laterite. In 1981, 13 years after construction, a condition survey showed about the same level of distress in both types of pavement. Twenty years after construction, roughness measurements, in terms of the International Roughness Index (IRI, *16*), showed an average 4.6 m/km for the as-dug laterite pavement and 4.2 m/km for the cement-stabilized pavement (*17*). At these levels of IRI, the minor difference is not significant in terms of vehicle operating costs, thus indicating similar performance of the two base types.

3. A 12-km section of the Mataara-Gatura road in Kenya was built in 1973 and 1974, 6 km with a crushed-stone base and the remaining 6 km with an as-dug laterite base (*18*). The main characteristics of the laterite were as follows. The mean laboratory-soaked California Bearing Ratio (CBR) at 95 percent of maximum dry density (MDD), British Heavy Compaction Test, was 54 (21 to 96 range); the mean plasticity index (PI) was 18 (7 to 21 range); the mean percentage passing a 0.063-mm sieve was 28 (15 to 37 range); and the mean in situ CBR at the surface of the base beneath bituminous surfacing was 89 (47 to 209 range). The local climate is characterized by a Thorthwaite's moisture index (*19*) of 174, which defines a perhumid climate. Field investigations carried out in 1982 and 1985 indicated that the laterite performed marginally better than the crushed-stone base (*20*).

4. Many road sections in Australia built with as-dug laterites have been performing well in comparison with sections with crushed rock or cement-stabilized materials. The performance of these road sections is evident in many areas where laterites are available, such as the Eyre Peninsular region of South Australia.

5. The 1,440-km-long Cuiaba–Porto Velho road in northwest Brazil was paved in 1983 and 1984. This road has a section of about 130 km (Rio Marco Rondon–Igarape Grande) where as-dug laterite was used as base course; the remainder was built with crushed-stone base (*21*). A survey carried out in May 1990 indicated that the laterite section was in fair condition, whereas several other sections were in poor condition (e.g., Caceres–Corrego Dourado, about 360 km). None of these sections has been resurfaced since construction.

6. A 1-km road section (Luwawa Turnoff to Champhoyo Trial Section) was built in 1984 and 1985 as part of a 51-km road contract on Route M12 in the Viphya highlands of the Northern Province of Malawi (*22*). The base course of the trial section was built with as-dug nodular laterite showing a mean PI of 17 and a 4-day soaked CBR of 31 (mean in-situ CBR 5 months after surface dressing was 80). The local climate is characterized by a Thorthwaite's moisture index (*19*) of 82, that is, a humid climate. As observed by Grace (*22*), the trial length of pavement is performing as effectively as the adjacent lengths, which have more expensive crushed-stone bases (about four times the cost of the laterite), and Benkelman beam deflections measured in 1986 were marginally smaller on the laterite-based section. Laboratory studies of the base laterite, carried out by Toll (*23*), indicated that if the laterite was compacted satisfactorily (i.e., not less than 95 percent of MDD British heavy compaction), it would retain adequate strength, even when the moisture content was increased to the point of saturation.

In addition to these examples, many more roads built with as-dug laterite have performed well. Souza et al. (*21*) give a list of road pavements built in Brazil with as-dug laterite bases that have shown good performance. These roads carry a wide range of traffic volumes, up to a maximum of about 2,400 veh/day/lane, with a 30- to 40-percent proportion of heavy commercial vehicles. The roads also represent a wide range of environmental conditions, varying from the semiarid Brazilian northeast to the Brazilian central plateau to the rain forest areas of the Amazonic region.

Although the use of as-dug laterites for pavement base course is most frequently recommended for the lower end of low-volume road, the Brazilian examples indicate that these naturally occurring materials led to good performance of roads carrying relatively high traffic volumes.

LONG-TERM VARIATION OF LATERITE PAVEMENT DEFLECTION

As part of the Brazil-UNDP road costs study [a major road research project conducted from 1975 to 1982 (*24*)], Benkelman beam and Dynaflect pavement deflections were measured periodically on 116 paved road sections. Ten laterite sections within a radius of about 50 km from Brasilia were measured with Benkelman beams every 2 to 3 months from 1976 to 1980 (*25,26*). A summary of the test section characteristics is given in Table 1. All sections have base and subbase courses built with as-dug laterite. The PI value varied from 0 (nonplastic) to 20, and 19 to 32 percent of the base course material passed the 0.075-mm sieve (No. 200), which can be considered typical of laterite gravels.

TABLE 1 SUMMARY OF TEST SECTION CHARACTERISTICS

TEST SECTION	ROAD NUMBER	CONSTR. YEAR	AADT (vpd)	ESAL PER YEAR	SURFACE TYPE	FIRST CRACKING (year)	BASE COURSE		
							CBR	PI	P75
001	EPCT	1972	95	4,000	AC	1976	112	14	19
002	BR251	1970	280	25,000	DST	NC	77	13	32
003	BR020	1965	3,780	250,000	AC	BC	131	0	20
004	DF20	1976	110	8,000	DST	NC	134	12	24
006	BR040	1960	5,600	850,000	AC	1977	102	14	32
007	BR020	1966	1,150	75,000	DST	BC	76	18	26
008	BR020	1966	1,150	75,000	DST	BC	75	16	25
009	BR060	1958	3,200	350,000	AC	1976	83	20	26
010	DF08	1968	1,020	18,000	DST	NC	70	18	24
011	BR070	1971	1,090	140,000	DST	NC	71	18	30

Notes:
(1) AC: Asphalt concrete
(2) DST: Double Bituminous Surface Treatment
(3) Section 006 was overlaid in 1976
(4) Section 009 was overlaid in 1968
(5) ESAL: Equivalent single axle loads
(6) NC: No cracking during observation period
(7) BC: Cracking started before observation period
(8) CBR: In situ California Bearing Ratio
(9) PI: Plasticity Index
(10) P75: Percent passing sieve no. 200
(11) AADT: Average annual daily traffic

The time series deflection measurements indicated a decrease of deflection over time. Results of linear regression run on data from each section, given in Table 2, indicated that deflection decreased on eight of the sections, increased slightly on Section 002, and remained unchanged on Section 009. On the average, the rate of deflection decrease was 0.02 mm/year on sections 008 and 011, 0.03 mm/year on Sections 001, 003, 004 and 007, and 0.04 mm/year on sections 006 and 010. The different trend of sections 002 and 009 could not be explained; the data available indicate that age, laterite plasticity, and surface type and thickness are not significant factors. However, the decrease in deflection over time can be interpreted as a phenomenon of self-stabilization, as discussed in the next section.

The deflection histories of the eight sections that showed a decrease in deflection were all similar in trend. The history of Section 001 is shown in Figure 1. Deflections were measured at 40 points in each wheelpath of each section; therefore, each point plotted in the figure represents the mean of 80 deflection points. Local climate is characterized by a Thorthwaite's moisture index (*19*) of about 60 (humid climate). The deflection measurements obtained in the rainy season (average rainfall is about 1500 mm/year, mostly occurring from November through February) are indicated by a plus sign in Figure 1. Although the deflection trend is significant over time, no significant influence of the rainy season on deflection was observed.

The conclusion is that a seasonal correction factor for deflections in regions similar to the study area (i.e., the Brazilian central plateau) is not needed. Information from other sources indicates that this conclusion is valid in other tropical and subtropical areas, where seasonal variability is lower than that observed for typical flexible pavements in higher latitudes.

A similar conclusion was obtained for the variation of Benkelman beam deflections with pavement temperature. Plots of deflection versus temperature showed no significant trend, thus indicating that for relatively thin asphalt surfacings (up to 60-mm-thick layers), corrections for pavement temperature are not necessary. This conclusion is reflected in a model developed by Queiroz, Visser, and Moser (*27*), which only gives correction factors significantly different from one when the asphalt layer thickness is higher than about 60 mm.

SELF-STABILIZATION OF LATERITE PAVEMENTS

Laterites, being a product of weathering, may actually be forming in the ground at the time of excavation. If this process continues in the road, it may give rise to the phenomenon of self-stabilization (*28*).

TABLE 2 RESULTS OF LINEAR REGRESSION RUN ON DATA FROM EACH SECTION

TEST SECTION	AVERAGE CHANGE (mm/year)	STANDARD ERROR (mm/year)	R SQUARED	NUMBER OF OBSERV.
001	-0.034	0.005	0.62	26
002	0.016	0.008	0.13	29
003	-0.029	0.005	0.57	27
004	-0.026	0.005	0.50	27
006	-0.041	0.007	0.57	29
007	-0.026	0.007	0.34	27
008	-0.015	0.009	0.11	26
009	0.004	0.011	0.00	28
010	-0.040	0.009	0.45	27
011	-0.021	0.007	0.25	26

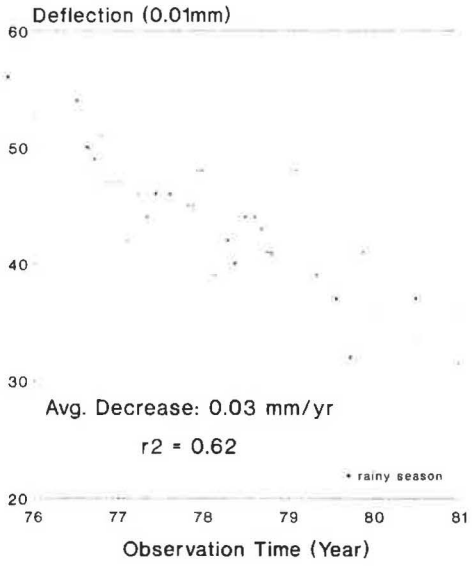

FIGURE 1 Section 001 deflection history.

Self-stabilization (or self-hardening) can be defined as a natural improvement in the strength of a pavement layer, not caused by traffic compaction or the addition of stabilizing agents (*29*). Although hard evidence of self-stabilization is not well documented, a test for potentially self-stabilizing laterites (i.e., petrifaction degree) was developed by Nascimento in the 1960s (*30,31*).

The decrease of Benkelman beam deflections over a 5-year period on eight different laterite pavements in Brazil, as described in the previous section, provides field evidence of laterite self-stabilization. Gains in the strength of a laterite gravel base over time have also been documented by the following examples.

1. In Western Australia, six laterite trial sections built on the Great Northern Highway in 1982 showed increase in strength over a 2.5-year observation period, as measured by a Clegg impact testing device (ITD) (*32*). The ITD measures the deceleration of a 4.5-kg hammer dropped from a height of 460 mm. The deceleration is indicated in *g*s, and 10 *g* is one impact value (IV) (*33*). The IV has been correlated with in situ CBR (*34*), and the increase in IV of the laterite base with time would correspond with decrease in deflection, had deflection tests been carried out.

2. In Malawi, a 1-km road section (Luwawa Turnoff–Champhoyo Trial Section) built in 1984 and 1985 indicated that the average of the in situ CBRs measured on the laterite base 17 months after surfacing was 16 percent greater than the average of the CBRs measured at the same locations 5 months after surfacing (*4*). The average relative moisture content increased from 0.93, 6 months after surfacing, to 0.96, 18 months after surfacing. This increase in moisture content would normally lead to a decrease in strength. However, the average increase of in situ CBR during this period indicates that self-hardening of the laterite is taking place.

3. In The Gambia, Benkelman beam deflections were measured on the South Bank Road (Brikama–Ba-Kalagi section), where as-dug laterite was used as base course, in 1981 and 1987 (*17*). Mean deflections were 0.96 and 0.91 mm in 1981 and 1987, respectively, resulting in an average annual decrease in deflection of about 0.01 mm/year.

ELASTIC BEHAVIOR OF LATERITE PAVEMENTS

In the analysis of pavement structural response to applied loads, unbound materials normally show nonlinear elastic behavior (i.e., their elastic moduli depend on the induced stress or strain), as pointed out by Haas and Hudson (*35*) and Yoder and Witczak (*36*). Medina and Motta (*37*) reported on several laboratory tests that also indicated nonlinear behavior of compacted tropical soils in Brazil.

To test in the field the nature of laterite pavement structural response to loads, an experiment was carried out on Sections 001, 004, and 006, which represent a wide range of traffic and age and two surface types (basic characteristics of these sections given in Table 1). Deflection measurements were taken on these sections with axle loads varying from about 3 to 12 tons (*27*).

Results of the deflection versus load analysis are presented in Table 3. The relationship obtained for section 006 is shown graphically in Figure 2. Deflections were measured at 10 points in each wheelpath of each section, for each axle load; therefore, each point plotted in Figure 2 represents the mean of 20 deflection points.

The results indicated a proportionality between deflection and axle loads, for a range of frequently occurring traffic loadings, thus indicating that the pavement systems (i.e., pavement layers plus subgrade soil) tested exhibit a linear elastic behavior. This behavior suggests the applicability of

TABLE 3 RESULTS OF DEFLECTION VERSUS LOAD ANALYSIS

Sec. 001 Regression Output:		
Constant		0
Std Err of Y Est		1.247
R Squared		0.995
No. of Observations		10
Degrees of Freedom		9
X Coefficient(s)	0.572	
Std Err of Coef.	0.005	
Sec. 004 Regression Output:		
Constant		0
Std Err of Y Est		1.839
R Squared		0.988
No. of Observations		8
Degrees of Freedom		7
X Coefficient(s)	0.427	
Std Err of Coef.	0.007	
Sec. 006 Regression Output:		
Constant		0
Std Err of Y Est		5.229
R Squared		0.950
No. of Observations		9
Degrees of Freedom		8
X Coefficient(s)	0.876	
Std Err of Coef.	0.022	

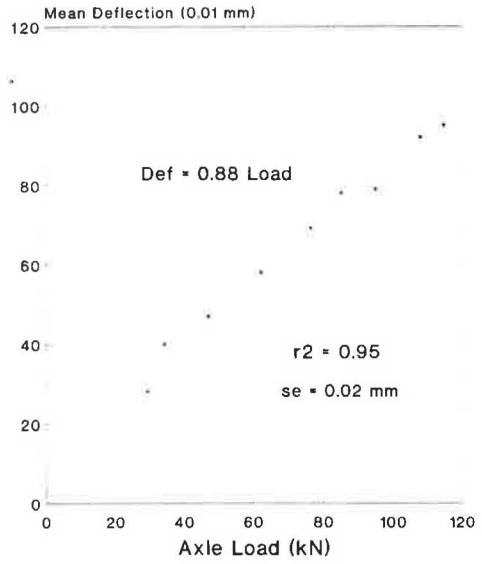

FIGURE 2 Deflection versus load, Section 006.

layer elastic theory to laterite pavements and indicates that correction for nonlinear behavior is not necessary. Although unbound pavement materials tend to show significant stress-dependent properties when tested in the laboratory, the results previously discussed indicate that they exhibit linear behavior as part of pavement structures.

Different axle loads have been used by various agencies to measure pavement deflection. The finding of proportionality between axle loads and pavement deflection indicates that a simple proportion can be used to convert between Benkelman deflections obtained with two different axle loads, making it simpler to compare and interpret deflection data from different sources.

RELATION BETWEEN BENKELMAN BEAM AND DYNAFLECT ON LATERITE PAVEMENTS

Pavement structural capacity can be evaluated through laboratory testing of the materials or directly, by in-place tests in the field. Field tests commonly used in a number of countries include Benkelman beam and Dynaflect. The Benkleman beam has had long and widespread use and is probably more familiar to pavement designers and engineers than any other deflection measuring device (*35*). The Dynaflect is an electromechanical device that consists of a dynamic cyclic-force (1,000-lb, 8-Hz) generator mounted on a two-wheel trailer, a control unit, and a sensor assembly (*38*).

Several equations relating Benkelman beam and Dynaflect deflections have been published, as reviewed by Paterson (*39*). Deflection data collected on Brazilian and Nigerian laterite pavements, using both Benkelman beam and Dynaflect devices in (quasi-) simultaneous measurements, are used here to investigate the relationship between these two types of deflection for laterite pavements.

The data from Brazil was collected as part of the Brazil-UNDP study (*24*) and included pavements in the Brazilian central plateau built with as-dug laterite bases and surface treatment and asphalt concrete surfacings (*26,40*). The Dynaflect parameter considered is the maximum deflection, that is, measured at the center of the applied load (or Geophone 1 deflection). Figure 3 shows the data points and the relationship obtained by regression analysis:

$$B = 23.4D \quad (1)$$

where B and D are Benkelman and Dynaflect deflections, respectively, in 0.01 mm. The r^2 value is 0.21, standard error of the B estimate is 16, the number of degrees of freedom is 85, and the t-statistic of the coefficient is 33. Adjusting a quadratic equation to the same data would yield a somewhat higher coefficient of determination (r^2) with still significant coefficients (at the 5 percent significance level):

$$B = 35.3D - 4.3D^2 \quad (2)$$

The r^2 value is 0.41, the standard error of the B estimate is 14, and the t-statistic values of the coefficients are 15 and 5, respectively.

The data from Nigeria were obtained from pavements built with as-dug laterite bases and asphalt concrete surfacings in Kaduna and Niger States (*41*). Figure 4 shows the scatter diagram and the linear regression equation derived from the data:

$$B = 35.2D \quad (3)$$

The r^2 value is 0.41, the standard error is 12, the number of degrees of freedom is 296, and the t-statistic of the coefficient is 64.

The relationships in Figures 3 and 4 show significant scatter and indicate that the two deflection measuring devices give different rankings of pavement strength in some cases. As Paterson (*39*) concludes, deflection measurements by Benkelman beam and Dynaflect are not directly interchangeable. The Dynaflect probably applies to the pavement a much lower

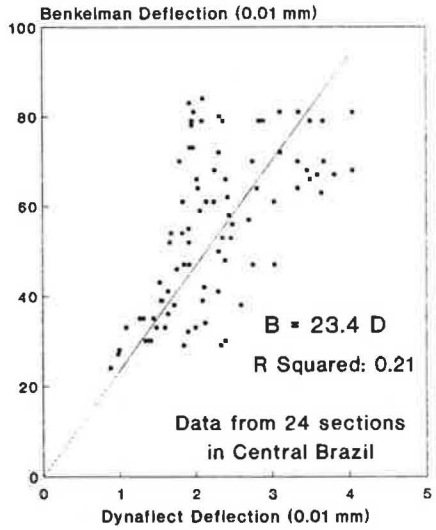

FIGURE 3 Relationship between Benkelman beam and Dynaflect deflections on laterite pavements in Brazil.

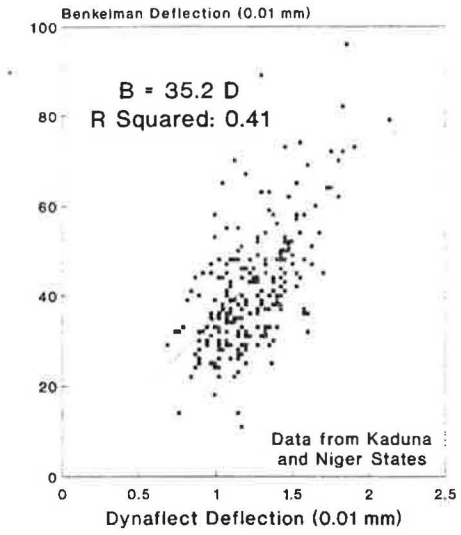

FIGURE 4 Relationship between Benkelman beam and Dynaflect deflections on laterite pavements in Nigeria.

load (with a geometry that is different from real vehicle axles) than commercial vehicles do, resulting in stress levels and depth of influence that are not representative of pavement reaction under heavy loads.

Independent analyses of Brazilian road deterioration data by Paterson (*39*) and Queiroz (*42*) showed that Dynaflect deflection parameters (e.g., maximum deflection, surface curvature index, base curvature index, and spreadability) were very poor explanatory variables (rarely significant) for predicting pavement deterioration, whereas Benkelman beam deflections had much stronger explanatory power. Benkelman beam and Dynaflect were the only deflection measuring devices used in the Brazil-UNDP study.

Input variables to the Highway Design and Maintenance Standards Model, HDM–III, (*43*) include Benkelman beam deflections. The practical implication of the previously discussed results for users of HDM–III (or similar road investment analysis models) is that efforts should be made to measure deflections using a device similar to that used to develop the model. Otherwise, accuracy of the analyses will be limited by the degree of interchangeability between the base and the adopted deflection measuring device.

RELATION BETWEEN SOAKED AND UNSOAKED CBRs

One of the main selection criteria for unbound base materials has been the soaked CBR (SCBR). However, this criterion has been questioned by several investigators, including Millard (*44*). In well-drained pavements, the field moisture content of laterite bases is normally well below saturation, even in wet climates, but more so in arid or semiarid areas. (Pavement engineers in general consider it axiomatic that poorly drained pavements behave poorly, irrespective of the quality of the base material.)

Autret (*2*) has shown that SCBR can be much lower than as-molded CBR, and that letting the compacted soil specimen dry for 4 days in warm air leads to even higher CBR. Analysis of CBR results from laterite bases obtained in 10 different asphalt pavements in Central Brazil (*25*) yielded the following relationship between SCBR and unsoaked CBR (UCBR):

$$SCBR = 0.78UCBR \quad (4)$$

The r^2 value is 0.36, the standard error of SCBR estimate is 23, the number of degrees of freedom is 18, and the *t*-statistic of the coefficient is 14. The scatter diagram and the derived relationship are shown in Figure 5.

In situ base course CBR values for the 10 previously discussed laterite pavements are also available (*25*). The field penetration test was performed similarly to the laboratory test by adapting a CBR press to a reaction truck, and the laterite base was kept at field moisture content. Analysis of the data showed erratic correlations between laboratory CBR values (soaked and unsoaked) and field CBR values, as expected for granular materials (*36*). Although no significant regression equation could be generated, the average relationships observed are given by the following equation:

$$ICBR = 1.02UCBR \quad (5)$$

and

$$ICBR = 1.27SCBR \quad (6)$$

where ICBR is the in situ CBR value of the laterite base.

On the trial sections at Mataara Gatura in Kenya, ICBR values carried out on the surface of the laterite base 10 years after surfacing showed a mean value of 92, whereas the mean 4-day SCBR value of the same material was 53 (*18*). Similar tests carried out at Luwawa Champhoyo in Malawi showed a mean ICBR value of 80 and a 4-day SCBR value of 31 (*22*).

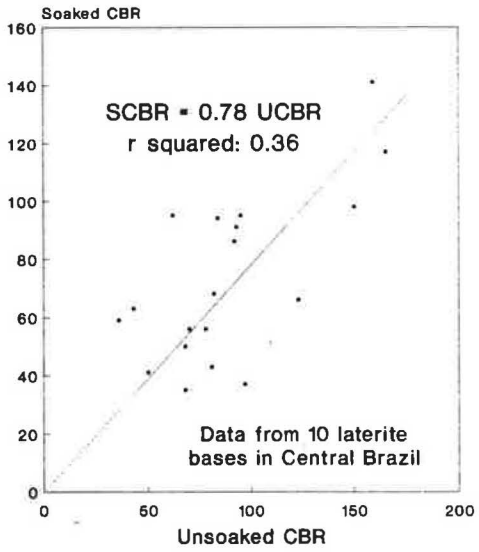

FIGURE 5 Soaked versus unsoaked CBR values.

LONG-TERM VARIATION OF PAVEMENT MOISTURE CONTENT

The possible reduction in pavement material strength caused by increased moisture content and the normal assumption that pavement moisture contents in flexible pavements will, at some time, reach saturated conditions, leads to the rejection of many naturally occurring materials (e.g., laterites) whose SCBR values are below performance requirements. However, three test sections of roads built in 1967 and 1969 in Australia indicate that the long-term pavement moisture content of pavements are more related to as-constructed, as-sealed moisture content than to environmental or climatic conditions. Beavis' evidence (*45*) indicated that pavement moisture content remained at, or very close to, as-sealed levels (Table 4).

Consequently, if pavements are allowed to dry out after construction and before sealing, and if the in situ moisture contents should remain at or near those levels as demonstrated in Australia, the acceptance criterion for pavement materials could more appropriately be the CBR value at the projected in-situ moisture content than the SCBR value.

That same series of tests indicated that the moisture contents in the middle 4.5 to 5.0 m of a 6.5- to 7.5-m sealed pavement remained substantially stable irrespective of climatic conditions; variations in moisture contents were limited to the 0.5 to 1.0 m of pavement nearest to the outer sealed edges. Beavis (*45*) concluded that the adverse effects of this increase in moisture content and subsequent weakening of the materials (especially the subgrades) under the edges of paved roads warranted "partial bituminous surfacing of shoulders . . . for a width of approximately 1 m outside edge lining."

Results from the Brazil-UNDP study (*24*) were consistent with those from Australia. In 18 laterite-base pavements in central Brazil, the moisture content of the laterite base was usually very close to optimum moisture content and was only 0.2 percentage points higher on average. Those field moisture contents ranged from 6.5 to 12.8 percent, and on a larger set of 25 pavements, up to 19.2 percent, in a humid climate of 30 to 100 on the Thorthwaite moisture index. Analysis of the materials showed that the field moisture content was related to the percentage of fines and the liquid limit of the material, as follows:

$$EMC = 3.75 + 0.167P075 + 0.136LL \quad (7)$$

where EMC is the field equilibrium moisture content (percent) and LL is the liquid limit (percent). The r^2 value is 0.49, the standard error of EMC estimate is 2.59, and the t-statistics are 1.7 and 2.5, respectively. PI and climate moisture index had no significant influence within the range observed.

PERFORMANCE

Intensive analysis of performance for a wide range of flexible pavement types, including natural gravel, crushed stone, and cement-stabilized bases, and wide ranges of pavement strength (0.2- to 2.0-mm Benkelman deflection under 80-kN axle load) and traffic loading [from 300 to 2 million equivalent standard 80-kN axle (ESA) loads per lane per year] have been reported by Paterson (*39*). This analysis was based largely on the Brazil-UNDP road costs study but also incorporated extensive validation from other countries and climates. When the performance of the lateritic base pavements from the Brazil-UNDP study were compared with other pavement types in these global models, the performance of lateritic flexible pavements proved to be similar to that of other pavement types of equivalent strength, and in some cases superior.

The time before cracking first appears is a function of both the time of exposure to weathering and the traffic loading relative to the pavement strength. The cracking behavior of 23 lateritic pavements observed in the Brazil-UNDP (*24*) study compared well with predictions from the following models (*39*):

Asphalt concrete:

$$TY = 4.5 \exp (0.14SNC - 0.2YE/SNC^2) \quad (8)$$

Surface treatment:

$$TY = 13.2 \exp (-17.1YE/SNC^2) \quad (9)$$

where

TY = predicted surfacing age (years) at first cracking,
YE = annual traffic loading (million ESA per lane per year), and
SNC = modified structural number of pavement strength.

TABLE 4 MOISTURE OF PAVEMENT MATERIALS WITHIN MIDDLE 4.5 m IN FOUR TEST SECTIONS IN AUSTRALIA

Site	Material	Moisture Content (% Mod. Optimum)			
		1967	1969	1971	1982
Merredin	Laterite Gravel	89	-	89	91
Loxton	Crushed Limestone/ Limestone Rubble	-	62	31	50
	Limestone Rubble	-	46	30	32
Lamerro	Sand Clay	-	40	39	51

Source: Beavis (45)

Asphalt concrete surfacings on lateritic bases tended to crack 27 percent earlier than expected when compared with all crushed-stone or natural gravel base types; these observed lives also tended to be rather short, but that was partly a function of the brittleness and susceptibility of those asphalt materials to weathering. Double-surface-treatment surfacings, however, which are more flexible than asphalt concrete, survived at least as long as expected on lateritic bases when compared with other base types, and the observed lives were much longer, ranging from 11 to 19 years. Once cracked, cracking progression rates were not significantly different from those for nonlateritic base types for both types of surfacing. Lateritic bases, therefore, tend to be less suitable for asphalt concrete surfacings than high-quality crushed-stone bases, and the fatigue strength of the asphalt material or the pavement strength must be used to compensate.

Roughness in flexible pavements has been shown to develop as a function of three types of components: (a) structural deformation, (b) surface defects, and (c) nontraffic or environmentally related deformation (*34*). When the roughness progression of the 25 laterite pavements was compared with the predictions of the general model, the predictions represented the observed behavior well.

BASE MATERIAL SPECIFICATIONS

Road-building practices, in general, have been heavily influenced by the prevailing conditions and pavement performance and behavior in temperate and cold climates. In those areas, pavement designers take great care to avoid the use of any materials in the pavement layers that are susceptible to the weakening effects of water and frost. Crushed rock and river-washed gravels are the predominant natural materials used for building roads in the higher latitudes. Because of the limited empirical or scientific research data from other parts of the world, the normally accepted material specifications for pavement bases in those areas are based on the pavement materials and their performance and behavior in the higher latitude countries. As pointed out by Toole and Newill (*46*), these specifications often result in acceptance of unsuitable materials and rejection of satisfactory materials. The consequences are that unanticipated financial penalties can be incurred or potential savings lost.

Some agencies have tried to disseminate more realistic material specifications for tropical regions. For example, 30 years ago the Portuguese National Civil Engineering Laboratory recommended increasing to 15 the maximum acceptable PI value for laterites to be used as base courses, provided that swelling does not exceed 1.0 percent (*47*).

Experience gained over the last four decades on the behavior and performance of laterite pavements indicate that as-dug laterites, which are outside the range of normally accepted specifications, have provided satisfactory performance as base courses without incurring increased road maintenance and vehicle operating costs. But examples of premature road failures attributed to particular materials and their deficiencies also exist. These failures are normally confined to a 1-m-wide strip of the pavement adjoining the edge of the surfaced pavement.

Previous sections presented some of the recorded field evidence from several countries, including Australia, Brazil, The Gambia, Niger, Nigeria, Kenya, and Malawi. On the basis of this evidence, more realistic specifications are proposed for the use of laterite bases, which would result in more widespread use of locally available natural materials in pavements. As long as special care is taken during construction to ensure adequate compaction and uniformity along the road, and if the bituminous surfacing is extended at least 1 m beyond the normal pavement width, the following specifications for using as-dug laterites in base courses could be adopted:

- Mean CBR value at 95 percent MDD British 4.5 kg compaction after 4-day soaking: not less than 40;
- Minimum CBR value at 95 percent MDD British 4.5 kg compaction after 4-day soaking: not less than 20;
- PI: not greater than 20;
- Percent passing 0.076-mm sieve: not more than 40; and
- Los Angeles abrasion of course grains: not greater than 60.

These specifications will allow the use of as-dug laterite from the majority of naturally occurring laterite gravel deposits. The main basis for relaxing existing specifications is the evidence of good performance and behavior of laterites that comply with the new specifications. However, additional factors also support the adoption of the relaxed specifications: (a) in situ material properties are the ones influencing performance, and ICBR values of laterite pavement layers tend to be higher than laboratory-soaked CBR values; and (b) laterite pavement strength tends to increase over time and under traffic.

The substitution of as-dug laterite for crushed stone or materials stabilized with cement or lime would result in substantial savings in construction cost without incurring additional road maintenance or vehicle operating costs. In a typical case, the savings in construction costs amounted to over $40,000/km in a 50-km contract. On the average, a crushed-stone base costs three to six times more than a laterite base.

CONSTRUCTION PROCEDURES

The inherent physical and chemical properties of laterites and the moisture behavior of laterite pavements under sealed surfaces are the factors that led to the recommendations for relaxing specifications for laterites used in pavements. However, the specifications, controls, and management of the construction process should not be relaxed. It is critically important that construction procedures be carefully defined and closely controlled and supervised. This control is particularly critical in quarry definition and working; laboratory testing and control; and site procedures for handling, spreading, compacting, grading, and making preparations for applying the seal coat.

In the quarry or pit, the depths of overburden and usable laterite, the quality of the materials, and the methods of working the pit must be carefully defined and specified. A grid of test pits and laboratory tests are essential to define the usable materials and the way in which the pit should be worked to prevent contamination of the usable material with unsuitable overburden, unacceptable laterite, and poor material in the

floor of the pit. Normally, deposits with less than 10 000 m^3 of usable laterite, in layers less than 0.5 m thick would be difficult to use in road works other than routine maintenance. To avoid contamination, average quarry yields would rarely exceed 50 percent of the total identified suitable material. In calculating material quantities, pavement compacted densities will be 20 percent higher than in situ quarry densities.

Procedures on the road must ensure that the material is worked in compacted layers between 75 and 150 mm deep. The issue of pavement layers is critical, and under no circumstances should thin make-up layers be added to compacted surfaces. Also, surface ponding must be avoided by ensuring a well-drained surface profile. Following compaction, as the surface dries out, cracks of about 1 to 2 mm, and occasionally up to 4 mm, may occur. These cracks will gradually disappear with continued watering, grading, and rolling. The use of traffic during this period could accelerate the process and more rapidly identify weak spots in the pavement that may have been caused by contamination of the laterites. These weak spots could then be repaired before sealing. After final compaction, the surface sweeping and cleaning preparation should expose the mosaic of laterite nodules to which the seal coat will be bonded.

ENVIRONMENTAL IMPACT

In addition to its economic and technical benefits, the use of as-dug laterites to build pavement bases may bring about positive environmental impacts, including (a) reducing air pollution because of the improved fuel efficiency of vehicles on the longer lengths of roads that can be upgraded or kept in good condition with the construction cost savings; (b) avoiding the use of energy that would be required for crushing and hauling rock; and (c) avoiding the importation of chemical products into the country, and the use of energy that would be required to process and haul materials to build chemically stabilized bases. Moreover, the increased use of materials such as laterites would enable developing countries to rely more on local resources and capabilities, thus reducing the requirements for foreign exchange. When opening the Seventh Conference of African Ministers of Transport, Adedeji (*48*) stated that every penny that can be saved, must be saved, and wastage of resources must be completely eliminated. The better use of local materials fits this policy statement well.

ADDITIONAL AND CONTINUING STUDIES

The proposed specifications can be adopted immediately for road construction and rehabilitation projects not exceeding 500 vehicles per day per lane, including up to 50 percent commercial vehicles, in areas at an elevation of not more than 2000 m, where the annual rainfall is not more than 2000 mm, providing the permanent water table is well below the road level. However, final conclusions on the precise relationship between laterite characteristics, environmental factors, traffic loadings, and pavement performance can not yet be drawn. Efforts are under way in several countries to further investigate the performance and behavior of laterites in pavement construction and rehabilitation. Trial sections are being built near Farafenni in The Gambia, and others are planned for construction on the Gabu-Buruntuma road in Guinea-Bissau. Additional work is being planned in Malawi, where a 100-km section of road is being upgraded to bituminous standards using naturally occurring lateritic base materials and the knowledge of construction procedures obtained from the Malawi Trial Section. The results of these trial sections will not be available for some years. However, many road sections have been built in the past with as-dug laterites and have not yet been systematically assessed and recorded. These existing sections could give an excellent sample from which actual performance and behavior could be analyzed. A systematic survey and study would provide a valuable laboratory for data of use to many developing countries.

Research authorities in several countries have recognized the useful properties of lateritic materials for road base construction, and studies and investigations have been undertaken in the United States, Australia, Kenya, Malawi, Gambia, Nigeria, Niger, Cote d'Ivoire, Brazil, England, France, Portugal, South Africa, and elsewhere. These investigations have shown beyond all reasonable doubt that naturally occurring lateritic materials can be successfully used as base materials for bituminous-surfaced roads. So far, however, very few highway authorities have amended their specifications to permit the use of these materials.

It is understandable that highway authorities, consulting engineers, and funding agencies are reluctant to relax specifications that have proven satisfactory over an extended period. However, these standard specifications, which have been developed in the temperate zones of North America and Europe, are overconservative or simply not applicable for conditions in tropical and semitropical regions. It is also understandable that any international authority would hesitate before assuming responsibility for such a research project. All research projects are by their nature open-ended. In view of the extensive work that has been carried out during the past 20 years, this research project could be finalized in a relatively short period of about 3 years. Few research projects would be likely to produce more benefits in less time for less cost.

The primary objective of further studies on this subject should be to provide hard evidence, in the form both of field and laboratory data, to substantiate the case for the use of suitable naturally occurring lateritic materials in base courses of bitumen-surfaced roads.

The next step would be for some international authority, such as the Permanent International Association of Road Congresses or the International Road Federation, to collect and examine the various studies and investigations that have been completed. This examination should suggest what additional work, if any, is needed and how and where it should be carried out. Finally, an authoritative document should be produced specifying the range of lateritic materials that can be used and the construction procedures that should be employed to ensure their satisfactory performance.

CONCLUSIONS

Data on the behavior and performance of bituminous-surfaced roads with base courses built from as-dug laterites has been

obtained from several countries. Although the use of these materials frequently does not comply with normal, internationally accepted specifications, many have shown excellent behavior and, in several well-documented cases, they have behaved marginally better than more costly alternatives, such as crushed-stone and cement-treated bases.

On the basis of the behavior of laterite pavements, new specifications for the selection of lateritic materials for base courses are advocated. These specifications would represent significant relaxation of previous internationally accepted limits. First, however, various completed studies and investigations need to be collected and examined. Additional work, if any, needs to be identified. Finally, an authoritative document should be produced, specifying the range of lateritic materials that can be used and the construction procedures that should be employed to ensure their satisfactory performance under a wide range of traffic loadings and environment. This is a role that some international agency should undertake.

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Comparison of the Impact of Various Unpaved Road Performance Models on Management Decisions

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Over the last 5 years, maintenance management systems for unpaved roads, such as the Maintenance and Design System and Highway Design and Maintenance System, have become increasingly available. Detailed data requirements have, however, deterred many potential users from implementing these systems, especially for low-volume roads. Furthermore, these systems are based on deterioration relationships obtained from a study in Brazil, and the applicability to other areas has not been confirmed. Therefore, local relationships with good predictability and low data requirements needed to be developed. During a recent study in southern Africa, a new set of relationships was developed for the prediction of roughness deterioration and gravel loss for low-volume unpaved roads over time. These relationships are much simpler than the previous ones, thus fulfilling local needs. The new relationships were compared with the existing models. The effects of the different models on management decisions about annual gravel replacement were examined by comparing the results of each model as it evaluated a typical unpaved road network. The blading frequency was evaluated by the blading cost and total cost, including vehicle operating costs. On the basis of these findings, conclusions about the potential necessity for upgrading can be drawn. Road authorities worldwide must consider the applicability of models developed in one environment to other environments and the potential impact on management decisions.

The necessity for improved management of road maintenance is becoming increasingly important as road networks become larger and funding is reduced in real terms. Although this is a worldwide problem, it is probably more significant in developing areas, where unacceptably rough unpaved roads result in very high vehicle operating costs. Major drains on the economy often result from increased consumption of fuel, tires, and spare parts, which are generally imported. Valuable foreign exchange can be saved by providing adequate roads. In many developing countries, spare parts are often almost impossible to obtain, and failure of an essential component that cannot readily be replaced may render an important vehicle inoperable for an indefinite period.

During the past decade, a number of maintenance management systems have been developed. The Maintenance and Design System, MDS (*1,2*), was developed from data obtained in Brazil, and the Highway Design and Maintenance System, HDM-III, (*3*) was developed by the World Bank on the basis of data collected from a number of countries. The Road Transport Investment Model, RTIM2 (*4*), was developed by the Transport and Road Research Laboratory (United Kingdom) and is based on deterioration models developed mainly in east and west Africa.

The impact of a previously presented set of models (*5*) developed in southern Africa has been evaluated. These models predict roughness deterioration and gravel loss over time. The input requirements of the different sets of models were compared. The impact that replacing the Brazilian models in the MDS with the new models would have on management decisions such as budget requirements for regraveling, routine grader blading, and upgrading, was determined.

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DESCRIPTION OF NEW MODELS

The new models were developed from data collected during the monitoring of 110 sections of road in the Transvaal province of South Africa and Namibia over a period of more than 3 years. A factorial experiment, including a wide range of traffic, climatic, and material properties, has been described fully by Paige-Green (*6*). The background and methods were summarized at the 4th International Conference on Low-Volume Roads (*7*). The development of the set of models has been discussed previously (*5*), but subsequent analysis indicated that particle size distribution and its method of calculation affected the regression model. The particle size distributions were standardized, assuming a maximum size of 37.5 mm (*8*), and the models were reevaluated. Although the statistics and variables in the model did not change significantly, minor changes to the coefficients were necessary.

Gravel Loss Prediction

The best model for the prediction of gravel loss developed during the southern African unpaved roads study (*6*) was as follows:

$$GL = D[ADT(0.059 + 0.0027N - 0.0006\,P26) - 0.367(N) - 0.0014(PF) + 0.0474(P26)] \quad (1)$$

[$r^2 = 0.84$; root mean square error (RMSE) = 5.3; $n = 703$; and $F = 619$]

where

GL = gravel thickness loss (mm);
D = time period under consideration in hundreds of days (days/100);
ADT = average daily traffic in both directions;
N = Weinert N-value (9), which ranges from 1 in wet areas to more than 10 in arid areas and incorporates annual rainfall;
PF = plastic limit × percentage passing 0.075-mm sieve; and
P26 = percentage passing 26.5-mm sieve.

The t values were 3.72, 6.64, −3.63, −9.72, −4.50, and 17.84 for the respective coefficients. The Brazil study (*1,2*) resulted in a number of gravel-loss prediction models, the following model being the most useful:

$$GL = D[1.58 + 0.366(G) + 0.083(SV) - 0.210(PI) + 0.0132(NC) + 0.0081(NT) + 420.45/R] \quad (2)$$

where

G = absolute value of grade (percent),
PI = plasticity index (percent),
SV = percentage of the surfacing material passing the 0.075-mm sieve,
NC = average daily car and pickup traffic in both directions,
NT = average daily truck traffic in both directions, and
R = radius of horizontal curvature (m).

HDM–III uses models developed from the data collected in Brazil to obtain the following gravel-loss prediction equation:

$$MLA = 3.65[3.46 + 0.246(MMP)(RF) + (KT)(ADT)] \quad (3)$$

where

MLA = predicted annual material loss (mm/year),
MMP = mean monthly precipitation (m),
RF = road rise plus fall (m/km),
KT = max{0; [0.022 + 0.969/57,300(KCRV) + 0.00342(MMP)(P75) − 0.0092(MMP)(PI) − 0.101(MMP)]}, and
KCRV = curvature (degrees/km).

One of the main advantages of the new model is its simplicity compared to the existing models. Aspects such as the vertical grade and horizontal curvature, which need to be averaged for a road link, are excluded from the new model. No estimate of the rainfall is necessary, and the necessary laboratory testing is minimal. All the parameters required can be easily obtained by relatively unskilled staff in unsophisticated laboratories.

Roughness Deterioration

The best model determined for southern African conditions (*6*) to predict the change in roughness in quartercar index (QI) counts per kilometer was as follows:

$$\ln R = D[-13.8 + 0.00022(PF) + 0.064(S1) + 0.137(P26) + 0.0003(N)(ADT) + GM(6.42 - 0.063\ P26)] \quad (4)$$

($r^2 = 0.22$; $n = 7005$; RMSE = 0.15; $F = 288$)

where

ln R = natural logarithm of change of roughness with time,
D = number of days since last blading in hundreds (days/100),
PF = plastic limit × percentage passing 0.075-mm sieve,
S1 = season dummy variable (1 for dry season, 0 for wet season),
P26 = percentage passing 26.5-mm sieve,
N = Weinert N-value,
ADT = average daily traffic, and
GM = grading modulus (sum of percentages retained on 2.0-, 0.425-, and 0.075-mm sieves divided by 100).

The t values for the respective coefficients were −14.52, 6.86, 6.75, 14.22, 13.66, 13.72, and −13.29.

If a value of 100 counts/km is predicted for the change in roughness, the actual value will, with 95 percent confidence, lie between 74 and 135 counts/km.

The Brazil study (*1,2*) involved a program of extensive roughness measurements that resulted in the following model for the change in the natural logarithmic value of roughness (LDQ) in terms of the QI value in counts/km:

$$LDQ = D\{0.4314 - 0.1705(T2) + 0.001159(NC) + 0.000895(NT) - 0.000227(NT)(G) + S[-0.1442 - 0.0198(G) + 0.00621(SV) - 0.0142(PI) - 0.000617(NC)]\} \quad (5)$$

where

T2 = surfacing type dummy variable (1 for clay, 0 for other),
NC = average daily car and pickup traffic in both directions,
NT = average daily bus and truck traffic in both directions,
G = absolute value of vertical grade (percent),
S = season dummy variable (0 for dry season, 1 for wet season),
SV = percentage of surfacing material passing the 0.075-mm sieve, and
PI = plasticity index of surfacing material (percent).

The model used in HDM–III was developed to constrain the tendency of these models to overestimate roughness at high levels under infrequent maintenance. The rate of roughness progression is decreased as roughness tends towards the maximum for a particular material. The result of this exercise was the following, somewhat complex, model:

$$QI(TG_2) = QIMAX_j - b[QIMAX_j - QI(TG_1)] \quad (6)$$

where

$QI(TG_1)$ = roughness at time TG_1 (QI counts/km);
$QI(TG_2)$ = roughness at time TG_2 (QI counts/km);
TG_1, TG_2 = time elapsed since last grading (days);
$b = \exp[c(TG_2 - TG_1)]$, $0 < b < 1$;
$c = -0.001[0.461 + 0.174\,ADL + 0.0114\,ADH - 0.0287(ADT)(MMP)]$; and
$QIMAX_j = \max[279 - 421(0.05 - MGD_j) + 0.22C - 9.93(RF)(MMP); 150]$ (for Section j)

where

ADL = average daily light vehicle traffic (<3500 kg) in both directions;
ADH = average daily heavy vehicle traffic (≥3500 kg) in both directions;
ADT = average daily vehicular traffic in both directions;
MMP = mean monthly precipitation (m/month);
RF = average rise and fall of the road (m/km);
C = degree of horizontal curvature (degrees/km);
MGD_j = material gradation dust ratio defined as $MGD_j = 1$ if $P425_j = 0$, and $MGD_j = P075_j/P425_j$ if $P425_j > 0$;
$P075_j$ = percent passing 0.075-mm sieve for Section j; and
$P425_j$ = percent passing 0.425-mm sieve for Section j.

In order to constrain the prediction model at high roughness values, the maximum value of the roughness is artificially limited to a QI of 150 counts/km. This value appears to be too low for the average unpaved road in rural and developing areas in southern Africa, although it may be defined as an acceptable limit before blading is necessary.

Roughness After Blading

The change in roughness needs to be related to a datum for use in a maintenance management system. For this purpose, a model to predict the roughness after blading (the starting point of roughness progression after each maintenance operation) was developed. The following model was the best obtained for southern African conditions (*6*):

$$LRA = 1.07 + 0.699(LRB) + 0.0004(ADT) - 0.13(DR) + 0.0019(LMS) \quad (7)$$

($r^2 = 0.62$; $n = 1601$; RMSE = 0.28; $F = 650$)

where

LRA = natural logarithm of roughness after blading (QI counts/km),
LRB = natural logarithm of roughness before blading (QI counts/km),
ADT = average daily traffic in both directions,
DR = dust ratio (ratio of percentage passing 0.075- and 0.425-mm sieves), and
LMS = laboratory-determined maximum size (mm), not greater than 75 mm.

The t values for the respective coefficients were 14.98, 45.99, 4.51, −2.67, and 3.23. The Brazilian model (*1,2*) for the roughness after blading is as follows:

$$LRA = 1.4035 - 0.0239(W) - 0.0048(SV) + 0.01694(PI) + 0.6307(LRB) + 0.1499(T1) + 0.3096(T2) + 0.00020(NT) + 0.2056(BS) - 0.01183(PI)(BS) \quad (8)$$

where

LRA = natural logarithm of roughness after blading.
LRB = natural logarithm of roughness before blading.
W = road width (m),
T1 = surfacing type dummy variable (1 if surfacing type is quartzite, 0 if other), and
BS = season during which blading occurred (0 if dry season, 1 if wet season).

HDM–III makes use of the following model:

$$QI_{(after)} = QIMIN_j + a(QI_{(before)} - QIMIN_j) \quad (9)$$

where

$QI_{(after)}$ = roughness after blading (QI counts/km),
$QI_{(before)}$ = roughness before blading (QI counts/km),
$a = 0.533 + 0.230\,MGD_j$,
$QIMIN_j = \max\{10; \min[100; 4.69\,D95_j(1 - 2.78\,MG_j)]\}$
$MG_j = \min(MGM_j, 1 - MGM_j, 0.36)$, and
$MGM_j = (MG075_j + MG425_j + MG02_j)/3$

where

$MG075_j = \ln(P075_j/95)/\ln(0.075/D95_j)$ if $D95_j > 0.4$, otherwise 0.3;
$MG425_j = \ln(P425_j/95)/\ln(0.425/D95_j)$ if $D95_j > 1.0$, otherwise 0.3; and
$MG02_j = \ln(P02_j/95)/\ln(2.0/D95_j)$ if $D95_j > 4.0$, otherwise $MG425_j$.

The Brazilian and HDM–III models require significantly more input, and the latter is especially cumbersome for developing areas. However, any of these models in association with the roughness progression models can be used to determine (a) the blading frequency necessary to retain the road roughness between upper and lower limits as required, or (b) economic efficiency by comparing the cost of maintenance with the road-user savings. However, the lower roughness limit is strongly dictated by the particle size distribution and plasticity of the wearing course material. The required roughness after blading may not always be achievable on roads with excessively oversized material or inadequate plasticity (i.e., those highly susceptible to the formation of rhythmic corrugations) through normal maintenance procedures.

Discussion of Results

An analysis of the applicability of the deterioration models for unpaved roads used in the different management systems indicated that the models developed outside southern Africa were not always applicable to southern African conditions (*6*). The average annual gravel losses for the sections monitored during the experiment were predicted as 13.0 and 21.8 mm by the Brazilian and HDM–III models, respectively; the actual measured value was 13.9 mm. The prediction of the

average roughness was even further out, being 96.1 and 66.2 QI counts/km for the Brazil and HDM–III models, respectively. The actual average measured value was 80.0 counts/km. The new model predicted values of 13.1 mm and 77.4 QI counts/km for the average annual gravel loss and average roughness. The Brazilian model predicts the annual gravel loss accurately but differs from the roughness prediction considerably, because the maximum size is not taken into account (*10*).

The Brazilian and HDM–III models are fairly complicated, necessitating the determination of a number of geotechnical properties, identification of the material types, and estimation of the average vertical grade (rise and fall) and average degree of road curvature, over the total length of the link. Because these models were developed mainly for use in developing countries, where computing facilities are often rudimentary (even at regional or head offices) and the skill levels of the road personnel may be low, the usefulness of the models, especially in remote areas, is questionable.

A significant aspect of the new models is their simplicity. The data required can be obtained quickly and cheaply using relatively unskilled labor. The models developed in this study eliminated the necessity to identify the material type and to estimate the average grade and curvature for the road link. Simple indicator tests requiring minimal equipment and only basic operator training are required for the input parameters for the model. The predictive capability, however, has not been diminished through this process. In fact, it has generally been improved for local conditions.

MDS

The MDS was originally developed by Visser (*1*) using data collected during the World Bank study in Brazil. The system has subsequently been improved and adapted for use on personal computers. An overview of its operation and use as a management aid has been fully described (*1,10*).

Management personnel generally desire certain information for the routine maintenance and upgrading of an unpaved road network. This information includes the following:

- How much money should be budgeted for regraveling, and what volume of material will be required annually?
- What routine blading budget is required?
- What are the consequences if the required budget cannot be provided?
- How many motor graders are required to perform the maintenance for the selected budget?
- How often should every link be bladed to ensure optimal economic allocation of the maintenance funds?
- Which roads are economically justified to upgrade to bituminous standard, and what funds are required for this?

All of these questions can be answered accurately and efficiently using the MDS.

GENERATION OF ROAD NETWORK

To evaluate the effect of the new unpaved road performance models on management decisions, a network of roads had to be selected. One of the criteria for this network was that all the information required for the old and the new performance models had to be available. In many instances, when performance models are applied to a network of roads, estimates of properties that are not very sensitive to the outcomes are made. Consequently, no readily available network of roads contained all the required information. A fictitious network was developed. This network consisted of all the test sections used to develop the new southern African performance models. A random number generator assigned a length to each link. In this way, a network of 2662.3 km comprising 77 links was developed. Traffic ranged from 20 to 333 vpd. The material, traffic, and environmental characteristics used in the analysis were those pertaining to the actual roads studied during the development of the models.

EVALUATION OF THE MODELS' EFFECTS ON MANAGEMENT DECISIONS

The effects of the old and new performance models on management decisions were compared. Gravel loss, maintenance budget, equipment requirements, overall network condition, and upgrading requirements were studied. Analyses were carried out on the original MDS using the Brazilian models (old MDS) and the new southern African models (new MDS).

Gravel Loss

The thickness of gravel lost, calculated according to the old MDS, ranged from 5.0 mm (the minimum permitted) to 27.6 mm; the gravel loss calculated by the new MDS ranged from 5.0 to 22.1 mm. The total volume of regraveling material required was 306 000 m^3 according to the old MDS and 259 000 m^3 by the new MDS. The new MDS thus predicted that 20 percent less material needed to be replaced annually compared to the prediction of the old MDS. By using the new MDS in southern Africa, budget requirements for gravel loss would be reduced by 20 percent. At a gravel cost of R10 per m^3, the savings would be about R0.5 million. This outcome, which is different from that found by calculating the average gravel loss, is the result of the different length and traffic characteristics of the network.

Maintenance Budget Requirements

Total cost comprises the road-user cost and the road maintenance cost. The point where the total cost curve reaches a minimum is the optimum maintenance position. This means that one unit of saving in road-user cost is balanced by one unit of cost to maintain the road. At this point, the marginal benefit-cost (B/C) ratio is 1. By means of the MDS package, the minimum position was calculated for both models. For the old MDS, the optimum maintenance cost (OMC) was R5.62 million and the road-user cost was R74.3 million. For the new MDS, the OMC was R4.76 million and the road-user cost was R70.9 million.

These results corroborate the previous statements about roughness progression. The rate of roughness progression is

lower for the new model, which means that the network is in a better condition with less maintenance. Hence, the OMC and road-user cost are lower for the new model.

The aim of maintenance scheduling is to ensure that maintenance is applied in such a manner that the economic advantage is the same on all links. For a few links this can be done manually, but as soon as a network of roads is considered, mathematical optimization techniques have to be applied. In the MDS, dynamic programming techniques were used. The economic advantage, expressed as the marginal *B/C* ratio, was computed for a range of maintenance budgets. The plots for the old and new models are shown in Figure 1. At the OMC, the marginal *B/C* ratio is 1.

Public authorities do not normally obtain sufficient funds for maintenance to be applied at economic optimality. Public expenditure on maintenance generally lies between marginal *B/C* ratios of 3 and 5. For the purpose of this comparison, a marginal *B/C* ratio of 4.0 was selected. The required budgets for blading were R3.521 million for the old MDS and R2.898 million for the new MDS (i.e., a 20 percent savings). In both instances, the inability to supply this budget would result in additional road-user costs of at least R4 for every R1 in maintenance funds not provided. Again, the new MDS resulted in a 20-percent reduction of the predicted budget requirements over those of the old MDS.

Equipment Requirements and Network Condition

Because of the lower budget requirements of the new model, fewer motor graders were required. Using the old MDS, 15 graders were necessary to maintain the network optimally; only 12 were necessary on the basis of the prediction of the new MDS. Even though fewer graders were required, the overall condition predicted by the new MDS was better. The weighted average network roughness for the new MDS was 63.2 QI counts/km, whereas the prediction for the old MDS was 81.5 QI counts/km. Besides the lower predicted budget and equipment requirements and better road condition of the new MDS, significantly lower road-user costs would be incurred by the traveling public.

Upgrading Requirements

The MDS has the capability of determining when it is economically justifiable, in terms of total costs, to upgrade an unpaved road to paved standard. For a given road under consideration, the output is the year when paving is justified, for a range of construction costs. This permits evaluation of the sensitivity of the construction cost.

The same 16 roads were selected for paving by both sets of models during the following 10-year period for the construction costs used. Because of the slower deterioration of the road predicted by the new models, however, the time of paving was generally set a few years later than that of the old models for the higher construction-cost scenarios. This time schedule again represented savings to the road authority.

Discussion of Results

The economic advantages of maintenance management have been shown in a number of countries. However, the economic advantages of using the new models in the MDS for southern African roads are clear. On the network under discussion, annual savings of R1.1 million, made up of R0.5 million for regraveling and R0.6 million for blading, are possible.

Because this savings is on a network of only 2660 km, the potential savings on the numbered unpaved road network of

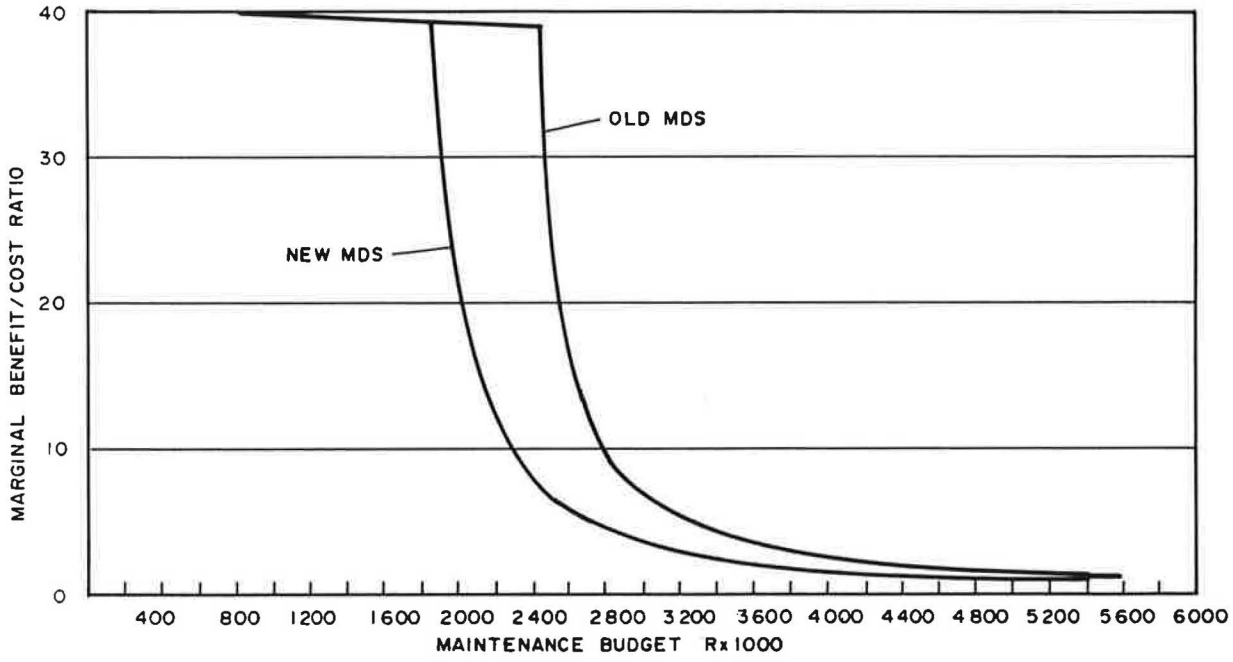

FIGURE 1 Graph for determining the appropriate maintenance budget.

140 000 km are enormous, assuming the network used in this study was a representative sample. The factorial experimental design used to select the original roads should ensure this.

CONCLUSIONS

The new performance-related models developed from southern African roads permit more confidence in the predictions of gravel loss and routine maintenance requirements for unpaved low-volume roads in southern Africa. The predictions also result in significantly lower budget requirements compared to those based on the models developed in Brazil. On a small network, savings of 20 percent, or R1.1 million, can be achieved. This amount alone is equivalent to the cost of the research project. The results of this work also show the danger of using models developed in a different environment.

ACKNOWLEDGMENTS

This paper is based on research carried out at the Division of Roads and Transport Technology and is published with the permission of the Director.

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Deterioration and Maintenance of Unpaved Roads: Models of Roughness and Material Loss

WILLIAM D. O. PATERSON

Empirical models of deterioration for the management of unpaved roads have been developed. Successive cycles of roughness progression and maintenance bladings are represented as a cyclic process reaching a steady-state pattern. One model predicts the minimum, maximum, and average roughness as functions of traffic volume, road gradient and curvature, physical material properties, rainfall, and the interval between bladings. In a second model, the rate of gravel loss is predicted from similar variables. Both were estimated from extensive data collected in Brazil, and both are compared with data from several other countries in Africa and South and North America, showing a good degree of transferability. The models have been incorporated in the HDM–III model for highway strategy evaluation.

An economic evaluation of alternative maintenance strategies for the management of unpaved roads is dependent on a reliable quantification of deterioration and the effects of maintenance. The frequencies at which maintenance blading and gravel resurfacing should be applied are dependent on the economic trade-offs between the costs of the maintenance and the benefits to be gained from reducing road-user costs. So, too, is the economic break-even point for upgrading an earth road with an all-weather gravel surface, or an unpaved road with a durable pavement. Empirical prediction models of deterioration, therefore, need to quantify the effective rates of deterioration, not only as a function of traffic, but also as a function of road geometry, material properties, climate, and the maintenance applied.

The following modes of deterioration are of primary relevance to the management of existing unpaved roads:

- Road roughness, which affects vehicle speeds and operating costs and is controlled by managing the maintenance blading of the surface so that the costs of blading are offset by the savings in operating costs; and
- Surface material loss on gravel roads, which makes them susceptible to rutting under traffic and raises the risk of losing passability in wet conditions, and is thus a primary determinant of the timing of regraveling operations.

In addition to these, rutting, passability, and looseness are modes of deterioration that are largely controlled at the design or construction stage through the selection of the surfacing material type and thickness necessary to support the traffic loading.

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The models of roughness progression and material loss are suitable for use in economic analysis models intended to evaluate the trade-offs between different maintenance and construction policies. These modes are considered in the World Bank's Highway Design and Maintenance Standards model, HDM–III (*1*), which is used in many countries with widely differing climates and materials. Therefore, the aim was to make the models as universal and transferable as possible. The work described is part of a broad study of road deterioration (*2*), which contains a fuller description of the individual modes and models.

For a management model, the life-cycle of deterioration and maintenance of unpaved roads can be depicted by the trends of roughness and surfacing material thickness over time, as shown in Figure 1. Roughness tends to increase rapidly under traffic at rates that may vary by season, and roughness is reduced by maintenance blading; therefore, the effect is a cyclic sawtooth trend. The effectiveness of the blading may depend on the material, its moisture content (and thus the time since the most recent rainfall), the type of motor grader or towed blade, the skill of the operator, and the roughness before maintenance. The average gravel surfacing thickness will be reduced gradually through whip-off and ingress into the subgrade. Regraveling may be triggered when a minimum thickness is reached, at which stage the traffic may be punching through to the weaker, underlying roadbed soil, resulting in major depressions and unevenness.

ROUGHNESS

Previous Work

The first model developed for predicting roughness progression on unpaved roads was a simple bivariate polynomial relating the roughness at any point in time to the cumulative number of vehicle passes since the last major reshaping or blading. This model was developed by the British Transport and Road Research Laboratory from data collected in Kenya from 1971 to 1974 (*3*). The model had a cubic form with different coefficients for each material type, determined by statistical regression from the data. A later Kenya (*4*) study used a similar model form but reported different coefficients for each material. The prediction curves from both studies, shown in Figure 2, displayed generally concave shapes of an increasing progression rate, but some displayed linear or slightly convex shapes. Different rates and stages of change were

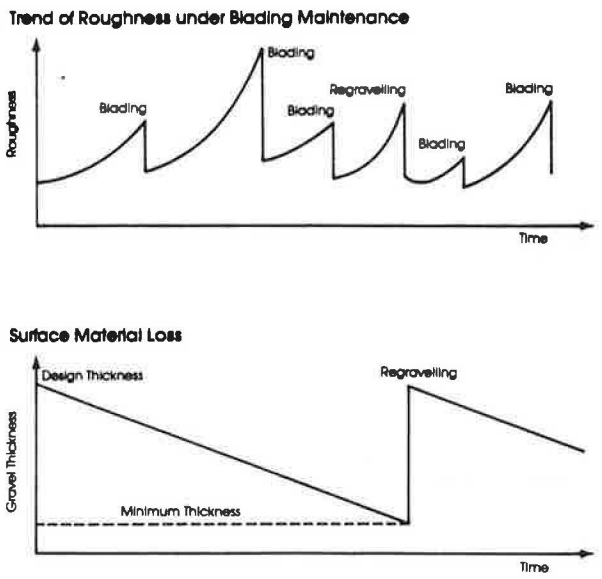


FIGURE 1 Trends of major deterioration modes of unpaved roads under maintenance.

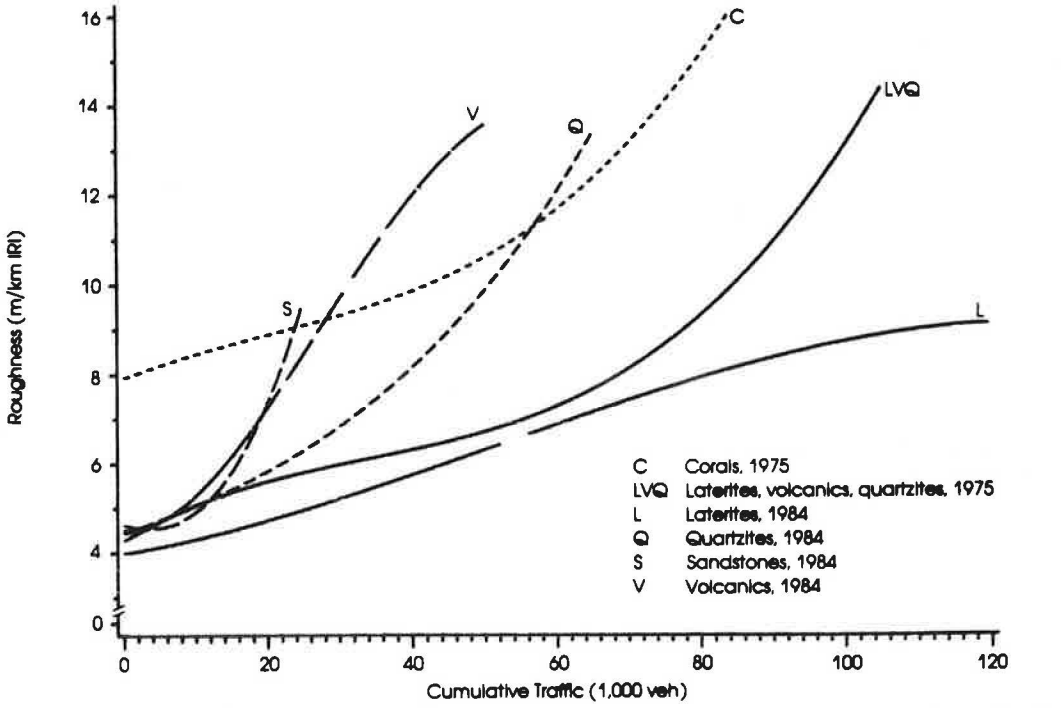
observed for different materials and between studies for a similar material. The result of blading was modeled as achieving a constant level of roughness, regardless of previous conditions. This level was 4.3 m/km on the International Roughness Index (IRI) for lateritic, quartzitic, and volcanic gravels, or 7.9 m/km IRI for coral gravels. Although material properties were measured, none were included in the models; thus, use of the models for other materials and other conditions was unreliable.

A wider range of material types, traffic volumes, and road geometries for unpaved roads was monitored in a major study sponsored by Brazil, UNDP, and the World Bank. The study involved road costs in Brazil from 1976 to 1981 (*5*). Visser (*6*) developed from the data an exponential (concave) model that was bounded by an exogenously imposed maximum roughness. A logit model form had also been estimated satisfactorily, but the exponential model was preferred for its computational simplicity. Visser also used an exponential form for modeling the effect of blading on roughness.

The inclusion of some material properties and rainfall in the Visser models was a major advance toward transferability, and some validation has been reported (*7*). A serious drawback, however, was that the progression model significantly overestimated the roughness progression on roads with low blading frequency, because of the exponential form, as discussed by Paterson (*2*). Because low maintenance frequencies are common in developing countries, improvements to the models were sought through different model forms. In subsequent analysis of the Brazilian data, these forms were estimated with a greater variety of material properties and rainfall data than was previously used.

Data Characteristics

Unpaved roads develop roughness through deformation by shear, mechanical disintegration, and erosion of the surfacing material, caused by traffic and surface water runoff. Roughness levels range from 4 to 15 m/km IRI, but higher levels occur when potholes, deep erosion gullies, or large depressions are allowed to develop, particularly on short sections.



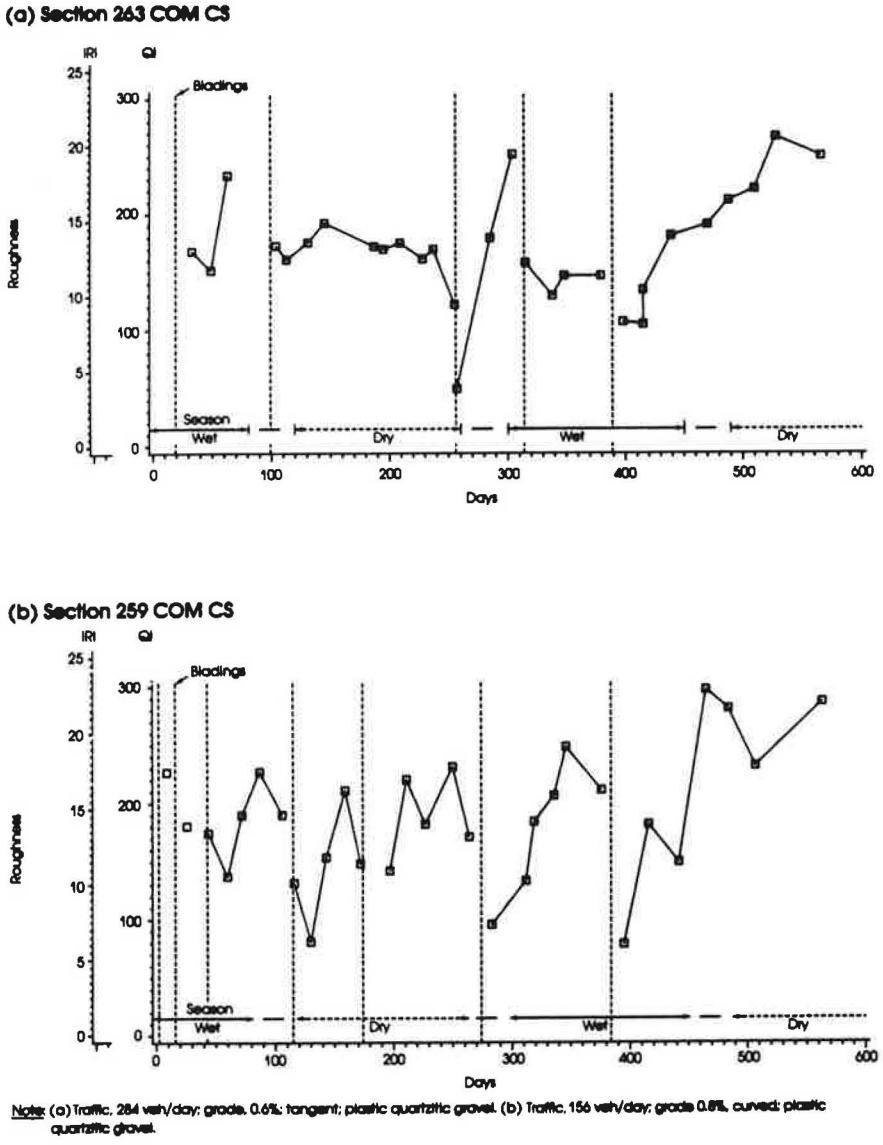
Note: Roughness conversion: RI = 0.0032 $RBI^{0.89}$ where RBI = mm/km TRRL Bump Integrator Trailer, and RI = m/km IRI.

FIGURE 2 Roughness progression for various materials in two Kenyan studies (*3,4*).

For the purposes of economic evaluation, the relevant roughness to be modeled is the profile in the prevalent wheelpaths of the traffic, since this influences the vehicle operating costs. As material properties, drainage, surface erosion, and the consequent location of the wheelpaths are variable, the roughness tends to be variable over time and the progression may not be regular.

Typical observations from the Brazil-UNDP study in Figure 3 show irregular patterns of both progression and blading effect. In some cases, an initial slight reduction in roughness results from a bedding-down effect under traffic, but a tendency to a convex trend shape is evident. Major differences in trend for any one road are evident between different cycles, some of which may be due to differences in weather. Also contributing to the irregularity is the high level of measurement errors (10 to 18 percent) found with response-type roughness instruments, which are the only type rugged enough to measure unpaved roads. For these reasons, large data sets must be used for model development, to take advantage of the error-minimizing effect of numerous observations. The Brazil-UNDP data set consisted of 8,095 observations on 48 sections over 2 years; the summary statistics of the data is presented in Table 1.

Because the roughness trends and blading effects are so irregular, and because the management of unpaved road maintenance is best defined by a policy of scheduled regular maintenance rather than one that is responsive to condition, a prediction model should be responsive to the main factors that will influence the frequency of maintenance, and it should accurately predict the average trends and conditions, rather than an individual cycle. The cyclic nature of conditions favors a steady-state approach in which the average conditions over a medium-term period of 1 to 4 years may be predicted. Thus, a steady-state model was postulated in which the roughness



Note: (a) Traffic, 284 veh/day; grade, 0.6%; tangent; plastic quartzitic gravel. (b) Traffic, 156 veh/day; grade 0.8%, curved; plastic quartzitic gravel.

FIGURE 3 Examples of roughness progression in Brazil-UNDP study (5).

TABLE 1 RANGE OF THE UNPAVED ROAD DETERIORATION STUDY IN BRAZIL (*5*)

Variable	Mean	Standard Deviation	Range Min.	Range Max.	No.
Gradient	3.8	2.6	0.0	8.2	48
Curvature on curved sections, km^{-1}	3.9	0.9	2.5	5.5	29
Road width, m	9.8	1.09	7.0	12.0	48
Maximum particle size, mm	18.3	9.8	0.07	39	48
Percentage passing the 0.075 mm sieve	36	24	10	97	48
Plasticity index, %	11	6	0	33	48
Average daily traffic (both directions)	203	-	18	609	48
Time relative to start of regravelling (days)	238	211	0	1,099	604
Roughness (m/km IRI)	8.7	4.2	0.8	32	8,095
Number of days between bladings	110	95	2	659	1,044
Last number of vehicle passes since blading in each blading period	16,080	17,880	63	136,460	1,044
Mean annual precipitation, mm	1571	56	1,506	1,746	48
Monthly precipitation, mm	131	-	0	608	48
Moisture index	59	16	35	100	48

Source: Compiled from GEIPOT (1982), study data files.

progression and roughness reduction by blading effects were solved simultaneously, producing a direct estimate of the average roughness applying over the successive cycles.

To achieve a simultaneous solution, a number of constraints must be assumed. First, the maximum roughness was postulated to be a function of the material and possibly the road geometry and weather, rather than a constant value. For instance, the roughness on positive grades is frequently less severe than the roughness on level sections, where potholes and depressions form if drainage and crown are poor. Second, the relevant roughness was assumed to eventually become asymptotic to the maximum roughness because of the combined effects of vehicles finding alternative wheelpaths and slowing down, as is partly evident in Figure 3. Ideally, this assumption could have used a sigmoidal curve to incorporate the concave road behavior that sometimes occurs immediately after blading, but none was amenable to a mathematical closed-form solution. Moreover, the major part of the progression phase is linear by all intents, and the assumed trend is essentially linear within the central part, which is the phase of interest. Finally, the roughness after blading was modeled as a linear function of how close it was to a minimum roughness. This minimum roughness is conceived of as the best condition that can be achieved by a motor grader on that particular surface material and is therefore expected to be a function of the material properties of the surfacing, and possibly of the foundation.

At a steady state, when the roughnesses before and after blading remain essentially constant for a regular blading maintenance at a given interval, the mathematical solution of these functions [which is detailed in another study (*2*)] gives the following equations:

Roughness progression:

$$RG_b = RG_{max} - p(RG_{max} - RG_a) \quad (1)$$

Roughness after blading:

$$RG_a = RG_{max} + q(RG_b - RG_{min}) \quad (2)$$

The long-term average roughness can then be expressed as a unique function of the limits and the vector functions p and q:

$$RG_{avg} = RG_{max} + (RG_{max} - RG_{min}) \times \frac{(1 - p)(1 - q)}{(1 - pq)\ln p} \quad (3)$$

where

p = $\exp(XT)$, such that $0 < p < 1$;
X = vector of explanatory variables that is to be estimated;
T = time between maintenance bladings, and $T > 0$;
q = vector of explanatory variables to be estimated, such that $0 < q < 1$;
RG_b, RG_a = roughness before and after blading, respectively; and
RG_{min}, RG_{max} = minimum and maximum roughness limits, respectively.

Using the comprehensive Brazil-UNDP study data, which covered 1,044 blading cycles lasting from 2 to 659 days on 48 sections with traffic volumes ranging from 21 to 609 vpd, the parameters in Equations 1–3 were estimated as follows:

$$p = \exp\{-0.001[0.461 + 0.0174(ADL) + 0.0114(ADH) - 0.0287(ADT)(MMP)]T\} \quad (4)$$

$$RG_{max} = 21.4 - 32.4(0.5 - MGD)^2 + 0.97KCV - 7.64GMMP$$

$$\text{and } RG_{max} \geq 12 \quad (5)$$

$$q = 0.553 + 0.230\,MGD \quad (6)$$

$$RG_{min} = 0.361D95[1 - 2.78(MG')]$$

$$0.8 < RG_{min} < 8 \quad (7)$$

where

ADT = annual average daily traffic in both directions (vpd);
ADL, ADH = annual average daily light and heavy vehicles per day, respectively (vpd);
MMP = mean monthly precipitation (m/month);
T = time between bladings (days);
KCV = average horizontal curvature [km^{-1} ($1\ km^{-1} = 180\pi$ deg/km)];
G = average absolute longitudinal gradient (percent);
D95 = maximum particle size of surfacing material, defined by size with 95 percent finder (mm);
MGD = dust ratio of surfacing material, defined as (P075/P425) when P425 > 0, = 1 otherwise; and
MG′ = min (MG, 0.36), where MG = mean material gradation, defined so that the optimum value of 0.5 is also the maximum, as follows: MG = min(MGM, 1 − MGM);
MGM = mean (MG075, MG425, MG02);
MG075 = ln(P075/95)/ln(0.075/D95) when D95 > 0.4, = 0.3 otherwise;
MG425 = ln(P425/95)/ln(0.425/D95) when D95 > 1.0, = 0.3 otherwise;
MG02 = ln(P02/95)/ln(2.0/D95) when D95 > 4.0, = MG425 otherwise; and
P075, P425, P02 = percentages of surfacing material finer than 0.075, 0.425, and 2.0 mm, respectively.

The gradation indices, MGM and MG, are thought to be a rational and useful basis for relating a gradation by its proximity to a Fuller type of maximum density gradation.

The respective t values for coefficients of p are 2.9, 5.9, 5.7, and 4.2; for coefficients of RG_{max} are 15.5, 5.0, and 1.0; for coefficients of q are 13.0 and 4.0; and for coefficients of RG_{min} are 7.1 and 4.6.

Roughness Progression

The rate of roughness progression, represented by the parameter p, was found to be a function of traffic and rainfall, with no significant effects of material properties being found. The maximum roughness, RG_{max}, was found to be a function of material properties, road geometry, and rainfall, which are all characteristics of the road that are essentially independent of age, traffic, and maintenance. RG_{max} represents the potential roughness that the road could reach under no maintenance. Although material properties were not explicit in p, which is the fraction of the roughness range, they are implicit in the rate of progression through the boundary RG_{max}, which determines the level and range of potential roughness for those materials and road characteristics.

The statistics show that the model is well determined, and the scattergram in Figure 4(a) indicates that the predictions fit the observed data very closely (the scattergram requires careful interpretation because of the high density of multiple observations on the line of equality). Much of the remaining variance derives from differences between cycles, for which there are very few explanatory parameters. For planning and management purposes, the aim is to select the best policies for different roads, and the scattergram in Figure 4(b) (which removes the between-cycle variance) shows that the model does this extremely well, with a significantly better fit, a standard error down to only 0.9 m/km IRI, and no bias apparent between the three material classes. The model thus takes good account of the major factors influencing policy decisions.

Predictions for the model are shown in Figure 5. The maximum roughness, in Figure 5a, is highest in rolling or hilly terrain, particularly in dry climates; it lowers as the rainfall or gradient increases, presumably because the gradient facilitates runoff of the surface water and because curvature accentuates transverse erosion (which influences roughness more than does longitudinal erosion of the roadway). Materials having a high dust ratio (i.e., clayey or poorly graded fines) appear to yield lower maximum roughness levels than well-graded materials, probably because earth roads tend to have lower roughness than gravel roads. The rate of progression, in Figure 5b, is dependent on traffic volume and is essentially linear in time or traffic over one-half to two-thirds of the roughness range. Light and heavy vehicles appear to have similarly damaging effects; the slight difference in the model is insignificant, which is a curious difference from Visser's models.

Effect of Blading

The model in Equations 6 and 7 indicates that the effectiveness of blading maintenance on roughness depends on the roughness before blading, the material properties, and the minimum roughness, RG_{min}. This minimum roughness, which is the minimum achievable by blading, was itself found to be a function of material properties, being least for fine materials and higher for coarse materials.

The model statistics and the goodness of fit indicate that the fit is as good as that of previous models. The variance derives largely from the operators' performance, because when the between-cycles variance has been removed for each section, the fit improves considerably to r^2 of 0.79 and the error reduces by 60 percent to 1.1 m/km IRI, or about the same level as for roughness progression. Lateritic and quartzitic gravels and earth surfaces are all well represented by the one model without undue bias. Details omitted because of space limitations may be found elsewhere (*2*).

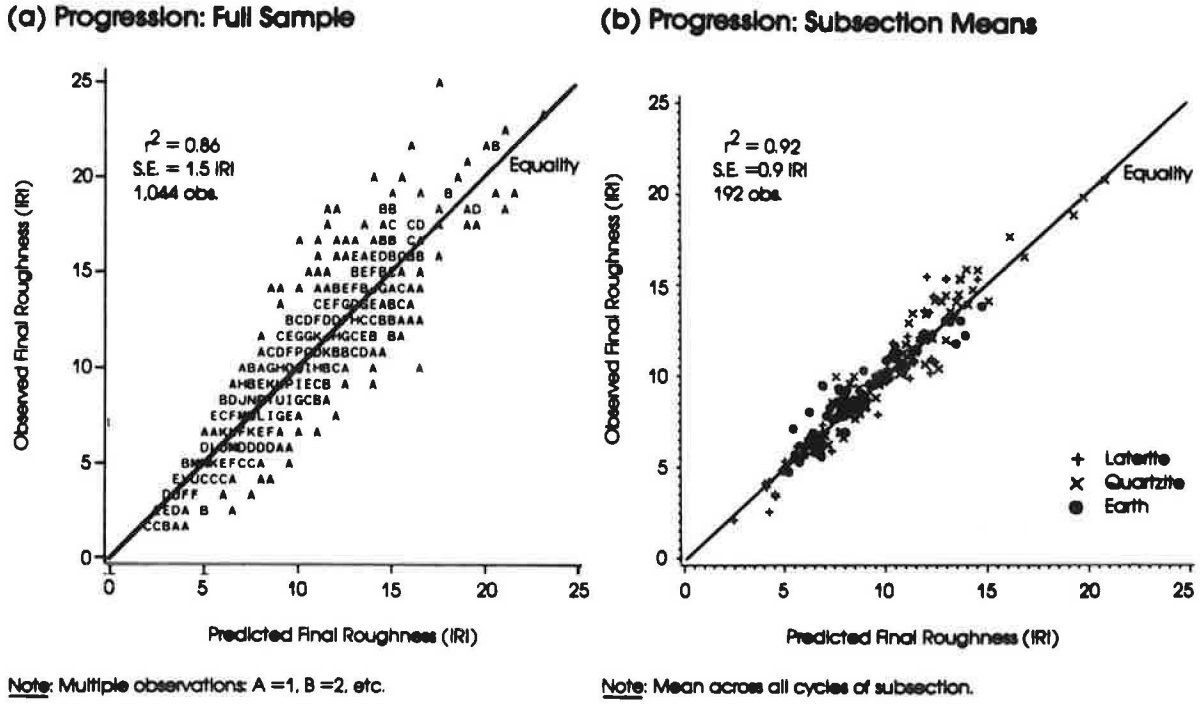


FIGURE 4 Goodness of fit of model for roughness progression on one road, with and without the between-cycle variance.

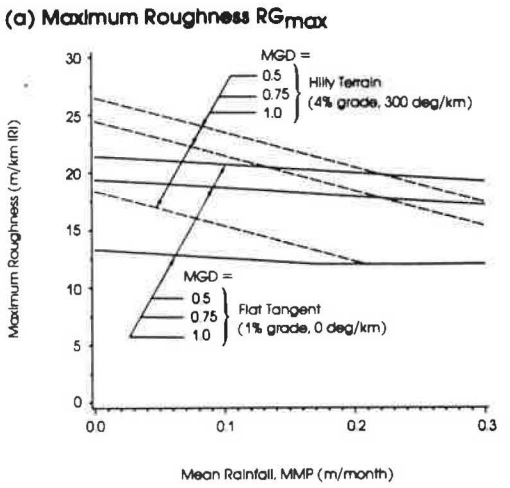


FIGURE 5 Predictions of maximum roughness and roughness progression rates relative to traffic volumes.

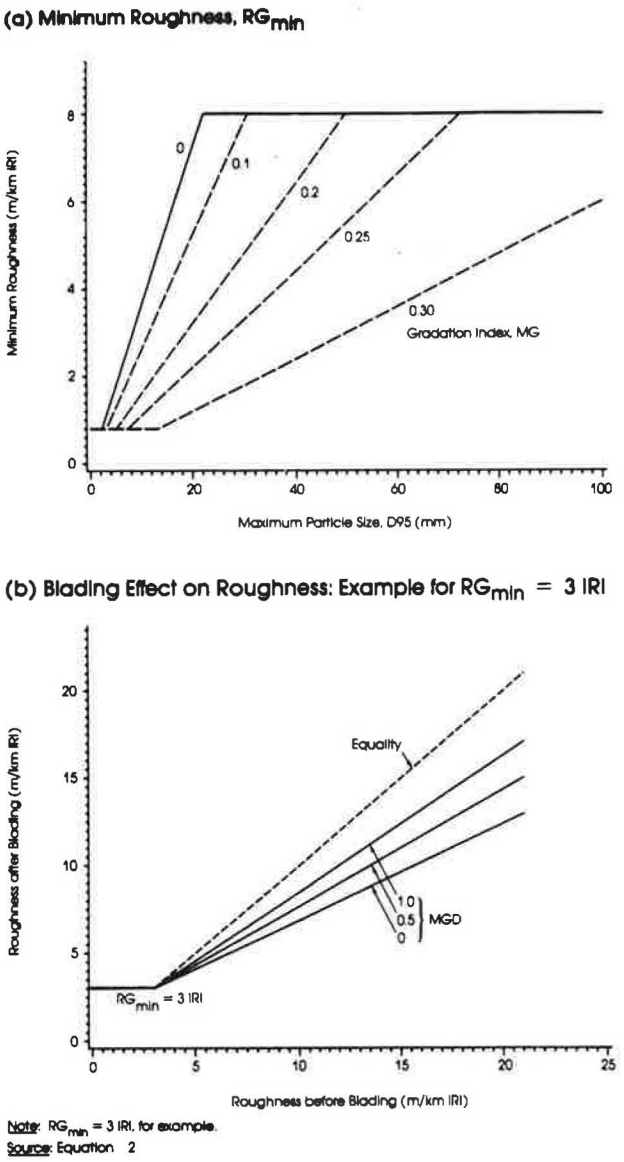


FIGURE 6 Predictions of minimum roughness and blading effects.

Typical predictions are shown in Figure 6. The minimum roughness, in Figure 6a, is quite sensitive to both the maximum particle size and the gradation of the material. Very low roughness levels can be achieved in fine or well-graded materials (e.g., less than 2 m/km IRI in all materials finer than 6 mm maximum size) or in well-graded materials (MG = 0.25 to 0.30) with a 20- to 30-mm maximum size. With poorly graded (MG less than 0.15) or very coarse materials, blading maintenance cannot reduce the roughness below 5 to 8 m/km IRI. The limits placed on RG_{min} were imposed to keep the predictions within the reasonable bounds of the inference space. The reduction of roughness achieved by blading, shown in Figure 6b, averages about 34 percent of the difference between the before-blading and minimum roughness levels, with only moderate sensitivity to material properties (ranging from 17 to 45 percent reductions as the dust ratio drops from 100 for clayey materials to 0 for sandy materials).

Predictions of Average Roughness Under Various Policies

The roughness trends under regular blading policies, as predicted by the model, are illustrated in Figure 7 for (a) a road under regular 90-day blading maintenance and different levels of traffic, and (b) a road under different (30-, 90-, and 360-day) blading policies and one level of traffic. The surfacing material is a medium-sized (D95 = 20 mm), slightly plastic gravel with high dust ratio (MGD = 0.80) and moderate gradation (MG = 0.20).

Increasing the traffic volume under a constant blading frequency has the dual effect of raising the average roughness and advancing the time at which the long-term average is reached from the minimum possible roughness. Increasing the blading frequency lowers the average roughness and advances the time at which the long-term average is reached. The

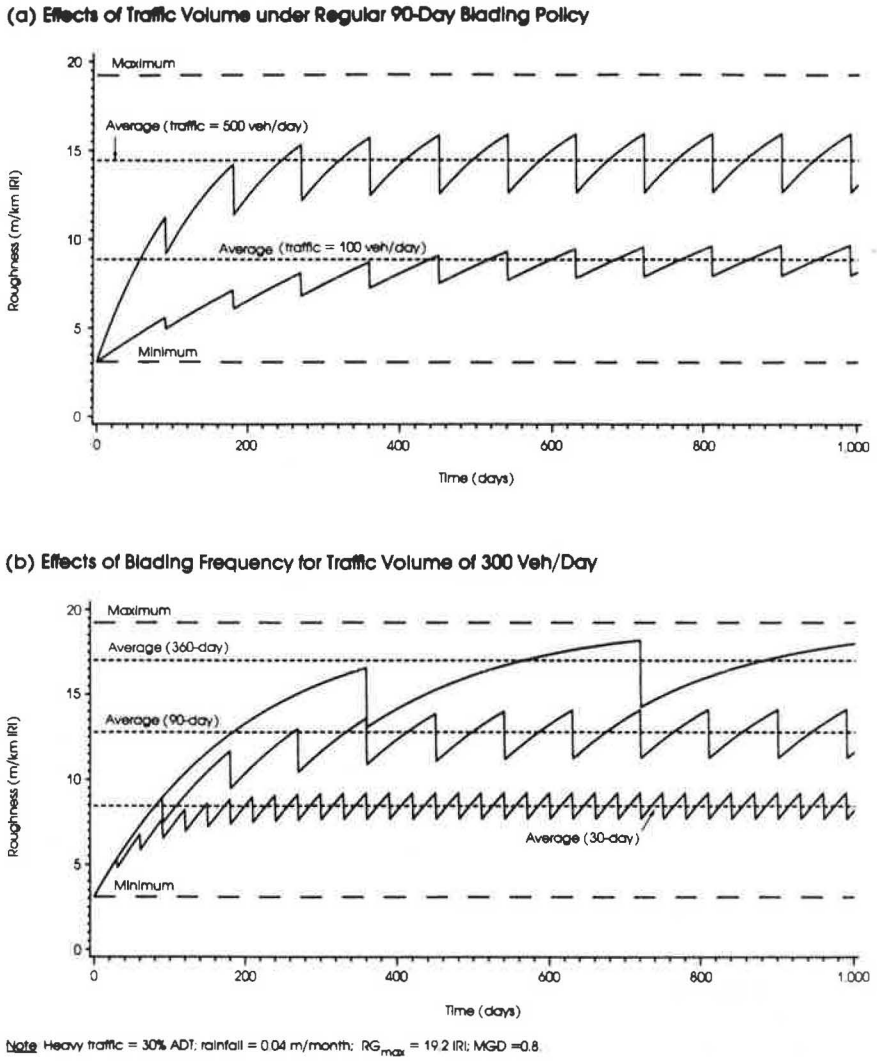
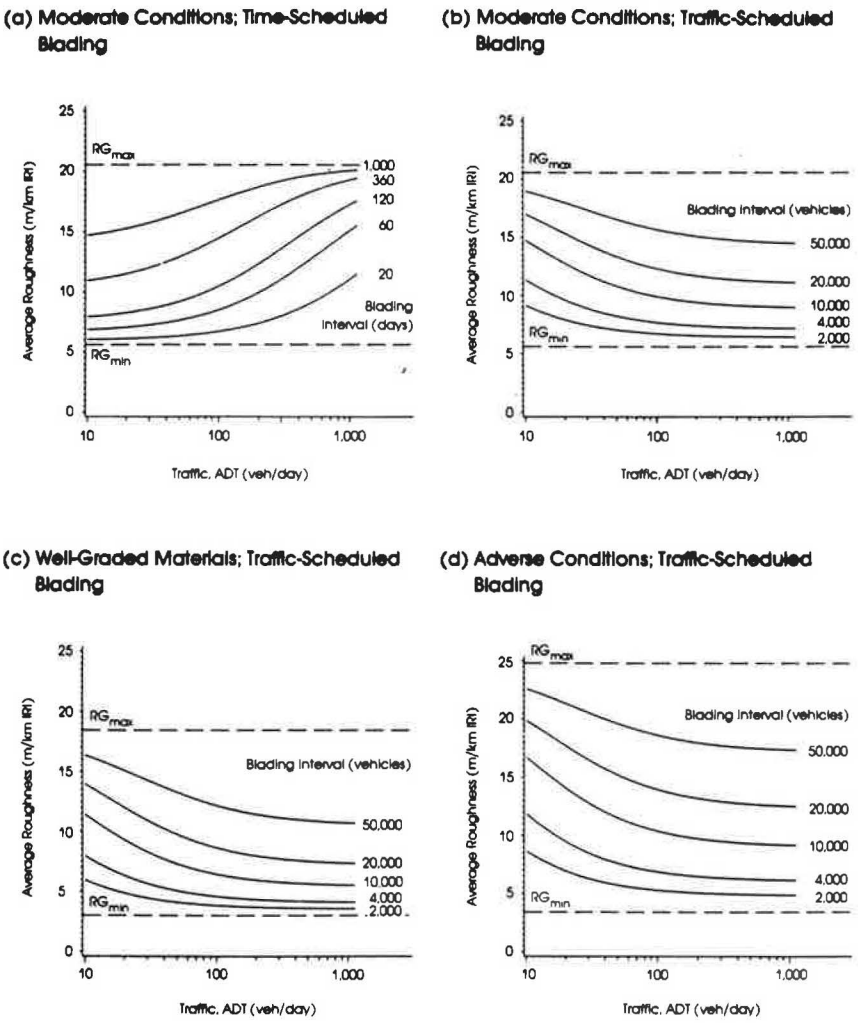


FIGURE 7 Predictions of roughness trends under regular maintenance.

roughness progression is essentially linear in most cases and only becomes noticeably convex at blading frequencies as low as once per year. Although the model is thus a reasonable approximation to reality for the long-term effects, the initial period of a slow rate of progression immediately following construction, recompaction, or deep blading (Figure 1) is not present, so that short-term effects may be poorly represented for such an initial cycle.

The average roughness predicted and the impacts of maintenance, traffic, and road characteristics are shown in Figure 8. The performance of an average case with moderate quality of materials, moderate climate, and moderate geometry, defined in Table 2, is illustrated in Figures 8a and 8b. The maintenance policy is scheduled by regular time intervals in Figure 8a, and in terms of the number of vehicle passes in Figure 8b. For traffic volumes of up to 200 to 400 vehicles per day (vpd) and bladings more frequent than every 120 days, the average roughness is relatively insensitive to either traffic volume or blading frequency. At higher traffic volumes or under less frequent blading, however, the average roughness levels increase more sharply. When the maintenance policy is defined in terms of vehicle passes, as in Figure 8b, the average roughness appears more clearly to be virtually independent of traffic volume, except for low volumes. Economic analyses using the model (*8*) indicated that a policy of blading at intervals of about 4,000 vehicles is close to optimal.

The influences of material properties and road or climatic characteristics are shown in Figures 8c and 8d. With materials of good gradation (even fairly coarse materials of the 50 mm size) and moderate conditions of climate and geometry, the levels of roughness predicted by the model are relatively low, as shown by Figure 8c. For poorly graded materials and adverse conditions, such as an arid climate or strong curvature, the range of roughness levels tends to be much wider and relatively high, as shown in Figure 8d. By way of comparison, the maintenance policies needed to meet a standard of 12 m/km IRI average roughness under a traffic volume of 200 vpd require regular blading at intervals of 54,000 vehicles for the well-graded materials in Figure 8c, 22,000 vehicles for the moderate conditions in Figure 8b, and 16,000 vehicles for the



Note: The characteristics of the materials, geometry, and climate are defined in Table 3.
Source: Equation 3.

FIGURE 8 Predictions of average roughness under various maintenance blading policies for various material and road geometry conditions.

poor material and adverse conditions in Figure 8d. These requirements correspond to blading frequencies of 270, 110, and 80 days, respectively, or a range of maintenance costs varying by a factor of about three.

Transferability

The transferability and validity of the models were tested on data from Kenya (*3,4*), Ghana (*9*), Ethiopia (*10*), and Bolivia (*11*). The blading effects model (Equations 2, 6, and 7) was tested on data from Ghana (*9*), which had sufficient information for the model. Six roads covered a wide range of material properties: very poor to very good gradation (MG = 0.13 to 0.43), fine to very coarse maximum size (9 to 80 mm), 0 to 19 percent plasticity, 1090 to 1910 mm/year rainfall, and 6.9 to 12.2 m/km IRI roughness before blading. Using the linear model (Equation 2), the average bias in the predicted roughness after blading was only +5 percent; the prediction error was 15 percent or 0.88 m/km IRI for the section-mean values. This accuracy is highly acceptable and even smaller than the estimation error of the original model (1.1 m/km IRI). By comparison, the Kenyan model prediction of 4.3 m/km IRI after blading is too optimistic for the Ghanaian data, which averaged 8.1 m/km IRI.

One test of the roughness progression model was on 186 km of lateritic gravel roads in the sub-Saharan country, Niger. With maximum stone sizes from 5 to 80 mm and roughness ranging from 5 to 11 m/km IRI, the predictions for the average roughness were accurate, within 7 percent of the observed averages, ranging from −6 to +7 percent (*2*).

Another test was on the data from 12 roads in four climatic regions of Ghana (*9*). These predictions, plotted in Figure 9, show a high correlation and close fit, with only a small bias causing the average of predictions to be 7 percent high. Individual predictions varied considerably (up to 40 percent different from the observed average); however, the overall prediction error was only 1.9 m/km IRI, which is similar to the error of the original estimation (Equations 4 to 7) and therefore as good as can be expected. Key data on the maximum

TABLE 2 PARAMETER VALUES USED IN EXAMPLES OF PREDICTED AVERAGE ROUGHNESS FROM THE STEADY-STATE MODEL

		Road and climatic conditions		
Parameter	Units	Moderate	Well-graded material	Poor material semiarid, hilly
G	-	3	3	3
KCV	km^{-1}	3	3	5
(curvature)	(deg/km)	(170)	(170)	(290)
MG	-	0.2	0.3	0.1
MGD	-	0.6	0.3	0.6
D95	mm	35	50	13
ADH/ADT	-	0.5	0.5	0.5
MMP	m/month	0.15	0.2	0.05
(rainfall)	(mm/yr)	(1,800)	(2,400)	(600)

Note: Parameter names defined in text.

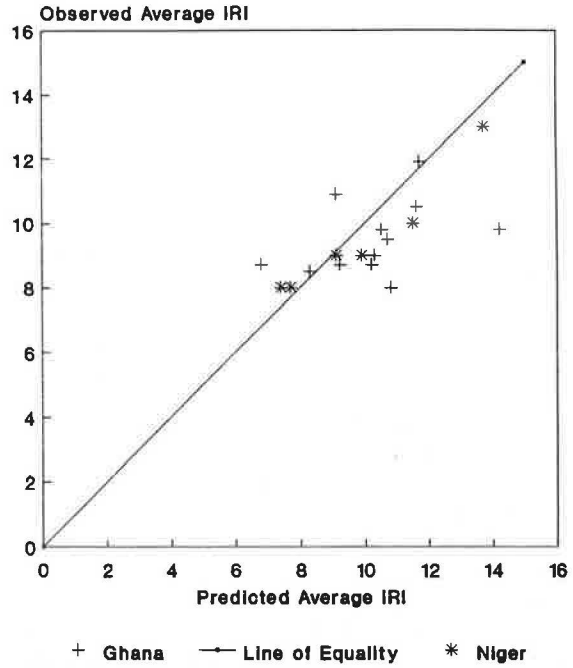


FIGURE 9 Comparison of observed and predicted average roughness and progression rates for Ghana and Niger.

stone size and blading timing were missing, which may partly account for the slight bias. The rate-of-progression predictions were consistent with the observations across climatic and material zones, although averaging 18 percent higher on a per-day basis, as shown in Table 3.

Analysis using data from Bolivia (*11*) indicated that the model predicted the rate of roughness progression well for a high traffic volume (1,700 vpd) and regular, repeated blading maintenance. However, for new surfaces during the initial stage of the first cycle after regraveling and compaction, the roughness rose slowly under traffic for the first 60 to 80 days, much more slowly than predicted by the model. At later stages, the model again predicted the rate of progression well both for high- and low-volume traffic (1,700 and 575 vpd). Binding and compaction of the surface may beneficially suppress roughness progression temporarily, and the model needs adjustment if it is to reflect that specific condition. Data and further discussion of this case are provided by Paterson (*2*, pp. 107–108).

In other studies of roughness progression, using data from Kenya and Ethiopia, the steady-state average roughness was usually difficult to determine from the reported data. The data, however, have a wide range of progression rates, ranging 7-fold on a per-vehicle rate from 0.05 to 0.37 m/km IRI (although most were in the range of 0.14 to 0.24 m/km IRI) per 1,000 vehicles, or 20-fold on a daily rate from 0.003 to 0.065 m/km IRI per day (*4*).

The progression model explained well most of the wide range of rates observed under different conditions through the material, traffic, and environment parameters, but it clearly applied primarily to roads under regular blading maintenance, even if at very low frequencies. The model apparently does not reflect the influence of maintenance compaction, which results in a much slower initial rise in roughness, as indicated most strongly by the Bolivian study. In the Brazilian study, bladings were performed frequently without special compaction, apparently resulting in a higher average progression rate; in the other studies, only one or at most five blading cycles were observed, and the roads generally began in a well-compacted and well-shaped state.

The progression model is, therefore, transferable in terms of material, traffic, and environment parameters, but it may tend to overestimate the rate of roughness progression when special treatment has been made to the running surface. The practice of compaction and providing cohesion to the surfacing material after major blading or regraveling, and the practice of providing a light running course of small stone, appear to reduce the rate of progression in a manner not reflected by the model. This effect was taken into account in the implementation of the model in the HDM–III model by the imposition of an arbitrary constraint on the first cycle. Future modeling work should quantify these effects. Future modeling should also possibly take a linear progression model form, as a compromise between the convex- and concave-shaped models, and focus on improving the estimation of the primary material and construction parameters that influence the performance.

MATERIAL LOSS

Regraveling is the major rehabilitation operation on unpaved roads, so the frequency required represents an important deci-

TABLE 3 VALIDITY OF PREDICTIONS FOR UNPAVED ROADS MONITORED IN GHANA

Road	Parameters[1]			Coefficents[2]		Avg. Roughness RG avg.		Daily Rate IRI/100 d		Per-veh. Rate IRI/100,000 veh	
Sect.	ADH	RG_{mn}	RG_{mx}	p	q	Pred.	Obs.	Pred.	Obs.	Pred.	Obs.
1G1	60	7.7	18	.80	.72	10.7	9.5	0.76	0.92	.18	.15
2G1	39	6.5	18	.84	.74	10.8	8.0	0.70	0.50	.15	.13
3G1	42	3.3	18	.83	.69	8.3	8.5	1.01	0.92	.27	.22
1G2	35	6.0	16	.86	.73	9.2	8.7	0.56	0.23	.21	.07
2G2	74	6.4	19	.80	.73	11.6	10.5	0.91	0.19	.10	.03
4G2	60	1.5	16	.82	.71	6.8	8.7	1.01	1.40	.24	.23
2G3	167	6.1	20	.61	.70	14.2	9.8	1.58	1.80	.09	.11
3G3	44	5.8	20	.83	.68	10.5	9.8	0.99	0.31	.23	.07
2G4	24	8.0	17	.88	.70	10.3	9.0	0.47	0.61	.28	.26
3G4	24	7.7	16	.88	.74	10.2	8.7	0.41	0.27	.35	.11
4G4	117	1.3	20	.74	.65	9.1	10.9	1.82	1.46	.17	.12
5G4	121	7.3	17	.74	.69	11.7	11.9	0.89	0.69	.09	.06
				S.E.		1.86					
				Average		10.2	9.50	0.92	0.78	.19	.13
				Avg. error		+0.70					
				Bias		+7.4%					

Notes. 1. Variables as defined in text. 2. Predictions based on maintenance of 2 bladings per year; roughness in IRI units of m/km (or ft/1000 ft) derived from car-mounted Bump Integrator data where 1 m/km ≡ 715 mm/km BI; Ghanaian data from Roberts (9).

sion. Gravel loss is defined as the change in average gravel thickness over a period of time. Gravel loss was evaluated for the interval between regravelings, which initiated a new analysis cycle, or from the time of the first observation until a regraveling occurred.

Three major factors identified as affecting gravel loss were weathering, traffic, and the influence of blading maintenance. Material properties and road alignment and width influence the gravel loss generated by each of these factors. In the Kenyan study (*4*), no seasonal pattern existed in the data; this also appeared to be the case for the Brazilian data. Furthermore, seasonal influences do not have any practical implications, because the agency responsible for regraveling wishes to know its frequency in terms of years, and seasonal influences are of secondary interest.

Estimation of Model

The following relationship was estimated from the Brazilian data for predicting the annual quantity of material loss as a function of monthly rainfall, traffic volume, road geometry, and characteristics of the surfacing material:

$$\text{MLA} = 3.65[3.46 + 2.46(\text{MMP})(G) + (\text{KT})(\text{ADT})] \quad (8)$$

where

MLA = predicted annual material loss (mm/year); and
KT = traffic-induced material whip-off coefficient, expressed as a function of rainfall, road geometry, and material characteristics.

$$\text{KT} = \max\{0; [0.022 + 0.969(\text{KCV}) + 0.00342(\text{MMP})(\text{P075}) - 0.0092(\text{MMP})(\text{PI}) - 0.101(\text{MMP})]\}$$

($r^2 = 0.313$; the standard error was 49 mm/year; the sample consisted of 456 observations; and the *t*-statistics were 3.1 and 2.6 for the coefficients of MLA and 3.7, 1.1, 3.9, 3.0, and 2.8 for the coefficients of KT.)

Figure 10 shows the predictions for a surfacing material of slightly plastic, fine, silty gravel, showing the effects of (a) traffic and rainfall for flat terrain and (b) rainfall and geometry for a traffic volume of 200 vpd. The effect of traffic volume

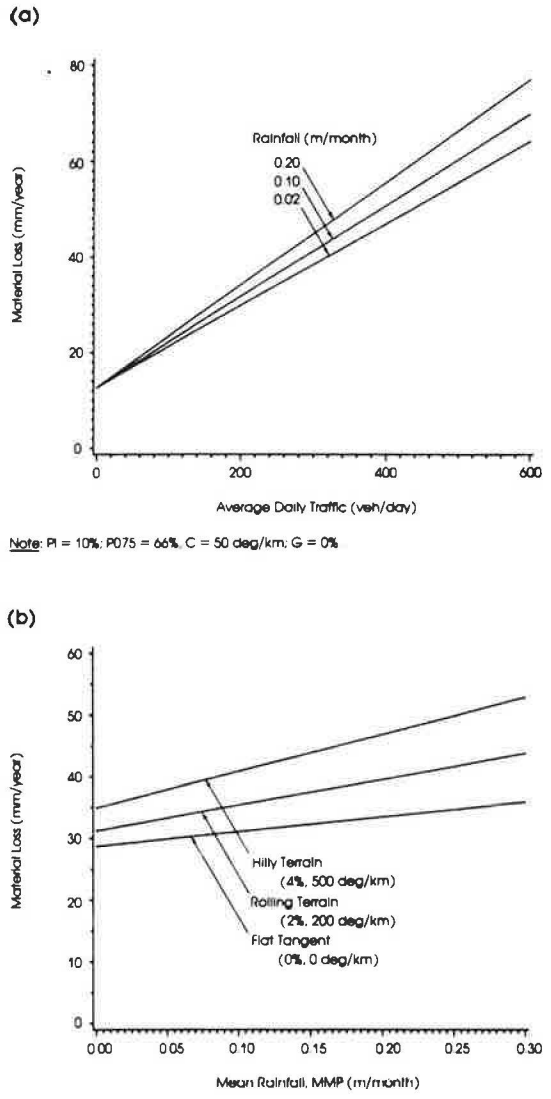


FIGURE 10 Predictions of surface material loss related to traffic, terrain, and rainfall: *a*, effects of traffic and rainfall on flat terrain; *b*, effects of terrain and rainfall under 200 veh/day.

was found to dominate the rate of gravel loss, with the annual rate increasing linearly by about 10 mm for every 100 vpd increase in ADT, and the average rate being about 30 to 40 mm per 100,000 vehicles, depending on other factors. Increasing the horizontal curvature increased the loss rate through whip-off under traffic, but the effect was not large and amounted to only a 20-percent increase over the full range of curvature from 0 to 5.5 km^{-1} (0 to 300 degrees/km). Rainfall also affects the loss rate, but by amounts that vary with gradient and with the fines' plasticity of the material; for materials with more than 50 percent fines, the loss rate is likely to increase, and for others it may decrease. Typically, rainfall may increase the loss rate by about 10 percent per 100 mm/month of rainfall. An influence of blading maintenance on the loss rate could not be identified.

Transferability

Comparison of four studies on gravel loss provided conflicting evidence on the effects of road gradient and rainfall, but a broad similarity on the effects of traffic, as presented in Table 4. Very strong rainfall effects were shown in the Kenyan model; an increase of from 1,000 to 2,000 mm of rainfall per year caused a 200 to 400 percent increase in loss rate, but this is now believed to be an overestimate caused by the analytical method used. In the Brazilian model, the comparable rainfall effects are only 7 to 10 percent, and in Ghana, the effects were slightly negative. In all studies, the effect of increasing the road gradient was to increase the loss rate by about 16 percent for each percentage point of gradient, but road gradient interacted with the amount of rainfall, increasing the loss rate slightly as rainfall increased. A slight reduction of loss under light rainfall is probably caused by suppression of dust loss, and loss because of erosion becomes significant under heavy rain.

Loss rates are thought to be best expressed on a per-vehicle basis, although the Brazilian model separates the traffic-induced and erosion-induced sources of loss. When the results of Table 4, including the U.S. Forest Service studies (*12*), are normalized to a common basis of mixed traffic with 50 percent heavy vehicles, the loss rates range from 25 to 45 mm per 100,000 vehicles for gradients of 0 to 3 percent. These rates are similar to the 30 to 70 mm per 100,000 vehicles reported by Jones (*4*) from a comparison of seven African studies, including data on Ethiopia, Cameroon, Niger, and the Ivory Coast in addition to those mentioned here. Only the Brazilian model contains estimates of the influence of material properties.

The best consensus on a universal model that can be deduced from these studies is the following general model, which incorporates the major traffic, gradient, and rainfall effects, but excludes material properties:

$$GL = 10^{-5}[30 + 180(MMP) + 72(MMP)(G)](h)(ADT)(T) \quad (9)$$

where

GL = average gravel or material loss (mm);
MMP = mean monthly precipitation (m/month);
G = average absolute gradient (percent);
ADT = annual average daily traffic (vpd);
h = proportion of heavy vehicles in traffic (fraction); and
T = time period (days).

The form of the new Brazilian model shown in Equation 8 is, however, believed to be the most representative of all the underlying mechanisms, and future research should seek to improve the universal model along such lines.

CONCLUSIONS

The approach to modeling the roughness of unpaved roads as a cyclic phenomenon of progression under traffic and

TABLE 4 COMPARISON OF GRAVEL LOSS RATES FROM VARIOUS STUDIES (*3–5,9,12*)

Study		Rate of gravel loss (mm per 100,000 vehicle units) for gradient (%) and annual rainfall (mm/yr)				Percentage heavy
		0%		3%		
Location	Units	1,000	2,000	1,000	2,000	vehicles
Brazil	V	30	32	39	43	50
Ghana	HV	(20-100)	(10-60)	40-160	(30-60)	-
	V	30	13	41	26	50
Oregon	AV	-	-	-	(240)	100[1]
	V	-	-		(40)	50
Kenya A	V	(7)	(21)	(12)	(60)	(35)
	V	10	30	17	86	50
Kenya B	V	10	29	16	82	-

1/ Assumed value.

Notes: V = per 100,000 vehicles; HV = per 100,000 heavy vehicles when light vehicles are present but not counted; AV = articulated logging vehicles, rated as AV = 3 HV = 6 V. -. Not available. ( ). Original data not adjusted for vehicle mix.

reduction under maintenance blading is a useful one for policy analysis. This approach provides a facility for predicting the average roughness levels under different maintenance blading frequencies and traffic volumes without having to resort to a year-by-year simulation process. The quantification of the influence of various material, road geometry, and rainfall factors in the model represents a significant advance in predictive capability and in the transferability of such a model.

The evaluation of the validity of the model against six independent studies in other countries indicated that the roughness after blading and roughness progression components explain the majority of effects and are sensitive to the major factors. However, there is an apparent tendency to overestimate the progression rate and the average roughness in the initial phase, when there has been compaction or other special treatment of the gravel surface. This tendency would result in a slight overestimation of the amount of maintenance input required for a given road, unless it is allowed for in the application of the model, as has been done in HDM–III, for example. The effects of compaction and transverse profile (crown) are outstanding issues that need to be studied and incorporated in future research and modeling efforts. A secondary issue is the model's shape: the convex shape of the progression curve concerns some, but the shape is virtually linear over the range of interest. The concave exponential model of Visser also has problems of overestimation under low traffic volumes. The most practical solution may be a linear progression model.

Material gradation, maximum particle size, and plasticity have all been shown to affect a road's performance significantly. A well-graded material, with small maximum size (but greater than 15 mm) and slight to moderate plasticity, yields the lowest roughness levels. The rate of roughness progression tends to be highest in dry conditions and is usually slower or sometimes negative (i.e., improving) under light to moderate rainfall. Significant variations in behavior on any given road occur from cycle to cycle of roughness progression and result from the operator's efficiency in the blading operations. These variations cannot be explained in detail by any of the models and must be regarded as stochastic variations.

The rate of gravel loss is influenced primarily by traffic volume, but the Brazil model indicates that small time- and rainfall/gradient-related elements that are independent of traffic also exist. Material, road gradient, and rainfall factors also influence the amount of loss caused by traffic, but only the Brazilian model quantifies the material property effects. Gradient has a generally positive correlation to loss rates, but evidence on rainfall influence is not entirely consistent across different studies and is probably fairly small, as indicated by this analysis of the Brazilian study.

ACKNOWLEDGMENTS

The major part of the work reported was conducted at The World Bank with support from the United Nations Devel-

opment Program. Thawat Watanatada contributed substantially to developing the steady state concept. Staff of GEIPOT and the Texas Research and Development Foundation team in Brasilia for the Brazil–UNDP Road Costs Study and of the Overseas Unit of the British Transport and Road Research Laboratory have been generous in their help with interpretation of data.

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Soils and Drainage

Field and Laboratory Evaluation of Geocomposite Drain Systems for Use on Low-Volume Roads

EDWARD STUART III, KENNETH S. INOUYE, AND JAMES A. MCKEAN

The Forest Service conducted an in-depth laboratory and field investigation to evaluate geocomposite drain systems for use on low-volume roads. The laboratory testing program was designed to determine the flow capacity of various manufactured systems when subjected to varying lateral loads and hydraulic gradients. Because no standard tests were available to evaluate geocomposite drain system performance under field conditions, a test system consisting of a large triaxial chamber and special plumbing was developed. Geocomposite test specimens were placed vertically in a 6- by 12-in. mold, which was then filled with a compacted silty soil. Changes in flow rates through the specimens were measured as both the gradient and lateral pressure were varied. A wide range in performance was noted among the different systems. At the lower confining pressures, the flow capacities were fairly similar. As the confining pressures were increased, the flow capacity of some drain systems dropped markedly. However, all products tested, except one, had a minimum flow rate of about 1 gal/min per foot of drain width when subjected to a confining pressure of 30 psi and a hydraulic gradient of 1.0. In conjunction with the laboratory testing, three field installations of prefabricated drain systems were instrumented. All three sites had piezometers in front of and behind the drain systems, and one site had an outflow recording device. Results of these installations validated the laboratory test results.

Proper drainage systems have long been recognized as the key to successful road performance. In the past, the U.S. Department of Agriculture (USDA) Forest Service has predominantly used aggregate drains (graded filter aggregate or coarse-graded aggregate completely enclosed in a geotextile). However, the use of aggregate drains on low-volume roads in remote areas or in difficult terrain has several drawbacks, including long aggregate hauls, the need for large storage areas, the difficulty of coordinating fill or retaining wall placement and drain installation, cumbersome construction, and labor-intensive installation.

The development of prefabricated or geocomposite drain systems in the early 1980s offered the potential for high flow transmitting capabilities, easy installation, and low costs. The Forest Service initiated a study to evaluate these products for use on various road projects. The study included both laboratory testing and evaluation of field installations.

GEOCOMPOSITE DRAIN SYSTEMS

Geocomposite drain systems, originally called fin drains or prefabricated drain systems, consist of a geotextile covering one or both sides of a core material. The geotextile permits water to pass through while retaining the soil, and the core transmits the water to the drain outlet. There are currently at least 15 manufacturers making more than 40 geocomposite drain systems. The primary differences in the products are the cores, which can vary from a dimpled polystyrene or polyethylene sheet to a nylon wire mesh to a polyethylene net, and the geotextile covering, which can be woven or nonwoven.

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TEST PROGRAM

A test program was developed to determine the flow capacity of the geocomposite drain systems when subjected to varying lateral loads. A preliminary examination indicated two potential factors that could greatly reduce flow capacity: (a) elongation of the geotextile into the core flow channels as a result of the soil pressures, and (b) compression or deformation of the core material by the lateral loads.

LABORATORY TESTING

Laboratory testing involved measuring flow through geocomposite drain systems by conducting constant head permeability tests with a 6- by 12-in. sample of the drain system placed vertically in a silty soil. The soil (AASHTO A-4) was compacted to a dry density of 85 lb/ft^3 (85 percent of maximum density as determined by AASHTO T99) around the sample in a 6-in.-diameter by 12-in.-high mold. The soil/geocomposite test specimen was then covered with a latex membrane and placed in a 12-in.-diameter by 40-in.-high triaxial chamber. The test system shown in Figure 1 was built to allow water to flow into the geocomposite sample at the bottom and out at the top under varying hydraulic gradients (0.3, 1.0, and 2.0) and confining pressures (5, 10, 15, 20, 25, and 30 psi). The ends of the geocomposite sample were open to permit an unrestricted flow of water. The type of soil used as selected to represent a worst-case condition, whereas the gradients and pressures were selected to represent typical field application conditions.

The test was performed by measuring the flow rate for each sample at various combinations of hydraulic gradients and confining pressures, maintaining the confining pressure until the flow rate stabilized. This procedure was used to ensure that the effects of soil, fabric, and core creep were included

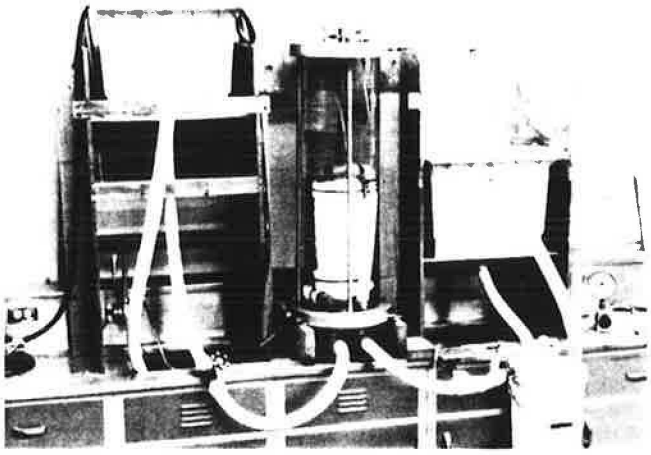

FIGURE 1 Laboratory test system.

in the results. The period required for this steady-state flow varied from a few days to many weeks. A minimum of two complete tests was performed on each product.

Graphs plotting the steady-state flow rate versus the confining pressure for each of the tested gradients were developed for each product. Graphs plotting flow rate versus time for a given gradient and pressure were also produced. Samples of these graphs are shown in Figures 2–5. Consolidated results for the tested products are shown in Table 1.

The results of the laboratory testing program show a wide range in performance among the different geocomposite drainage systems. Some products (e.g., Miradrain and Contech) have rigid cores, and their flow rates showed only slight decreases with increasing confining pressure. The decrease in flow (around 10 percent) is believed due to a reduction in the cross-sectional area of the geocomposite drain system caused by a combination of compression of the core and elongation of the geotextile into the flow channels. However, when the crushing strength of the core was exceeded, the flow rate dropped sharply. Figure 2 shows a dramatic decrease in flow for the Contech Stripdrain when the confining pressure was increased from 20 to 25 psi. Figure 3 shows a similar decrease in flow capacity for the Miradrain 4000 when the confining pressure was maintained at 30 psi. Other geocomposite drain systems have compressible cores, and flow rates for these products dropped markedly with increasing confining pressure. The Enkadrain, with its compressible mesh core, was especially susceptible (see Figure 2).

When the confining pressure was increased during the test, the flow rate usually stabilized within a few days. However, the slow crushing process of the Contech product caused the flow rate to gradually decrease over a period of months (see Figures 4 and 5). All other products, including those whose core was crushed, had a minimum equilibrium flow rate of about 1 gal/min per foot of drain width when subjected to a hydraulic gradient of 1.0 and a confining pressure of 30 psi. The Contech product exceeded this value at a confining pressure of 25 psi.

FIELD INSTALLATIONS

In conjunction with the laboratory testing, three field installations of geocomposite drain systems were instrumented for

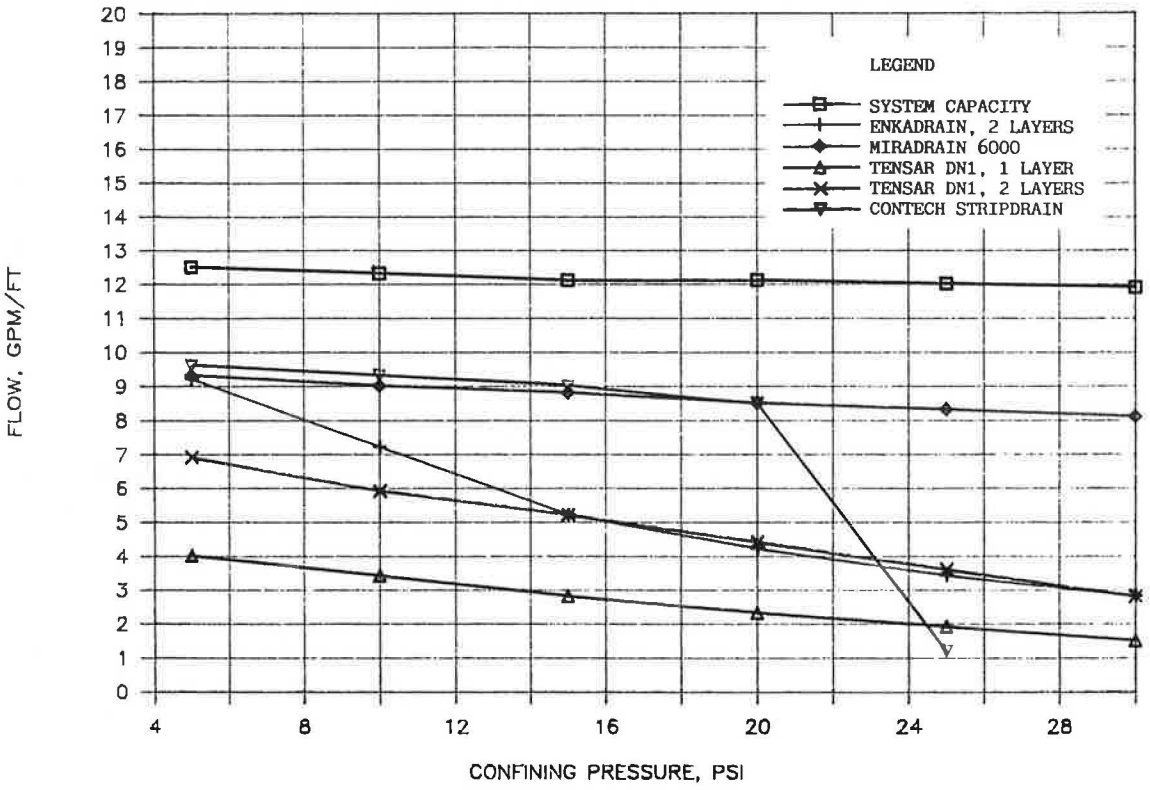

FIGURE 2 Equilibrium flow versus confining pressure: 1.0 gradient.

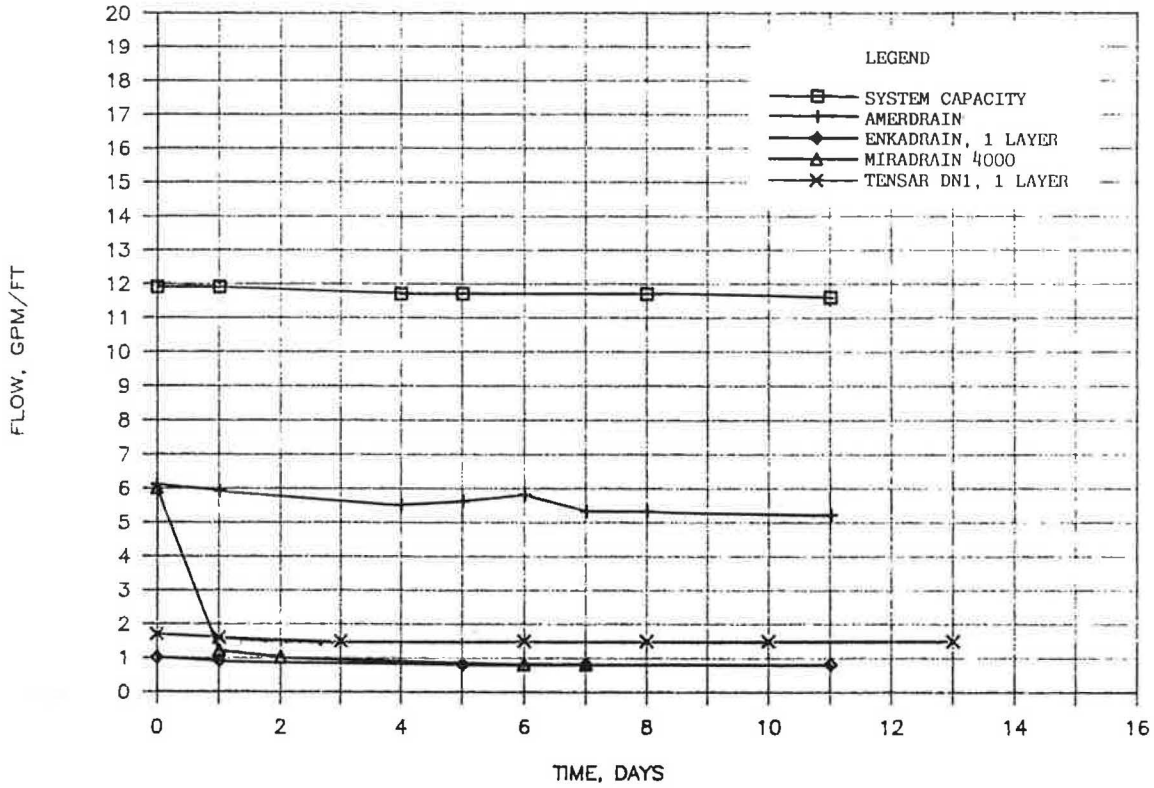

FIGURE 3 Equilibrium flow versus time (0–16 days, 30-psi confining pressure, 1.0 gradient): system capacity; Amerdrain; Enkadrain, one layer; Miradrain 4000; Tensar DN1, one layer.

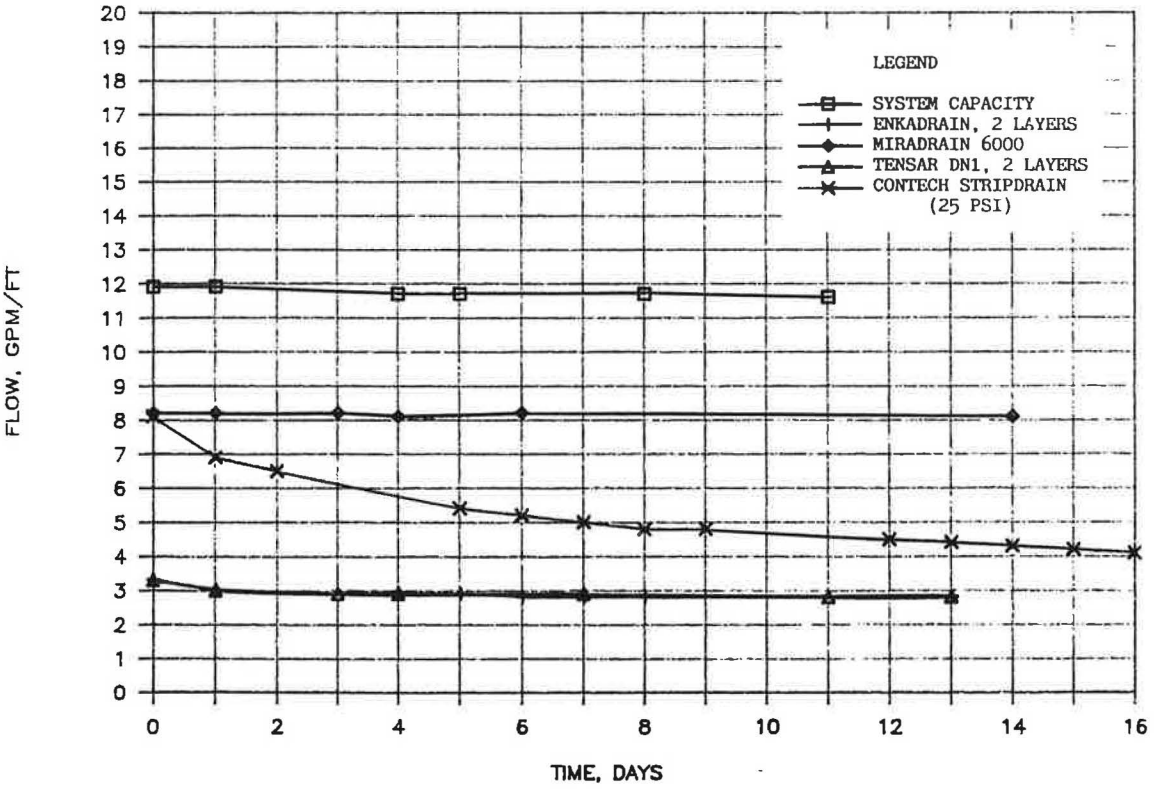

FIGURE 4 Equilibrium flow versus time (0–16 days, 30-psi confining pressure, 1.0 gradient): system capacity; Enkadrain, two layers; Miradrain 6000; Tensar DN1, two layers; Contech Stripdrain (25 psi).

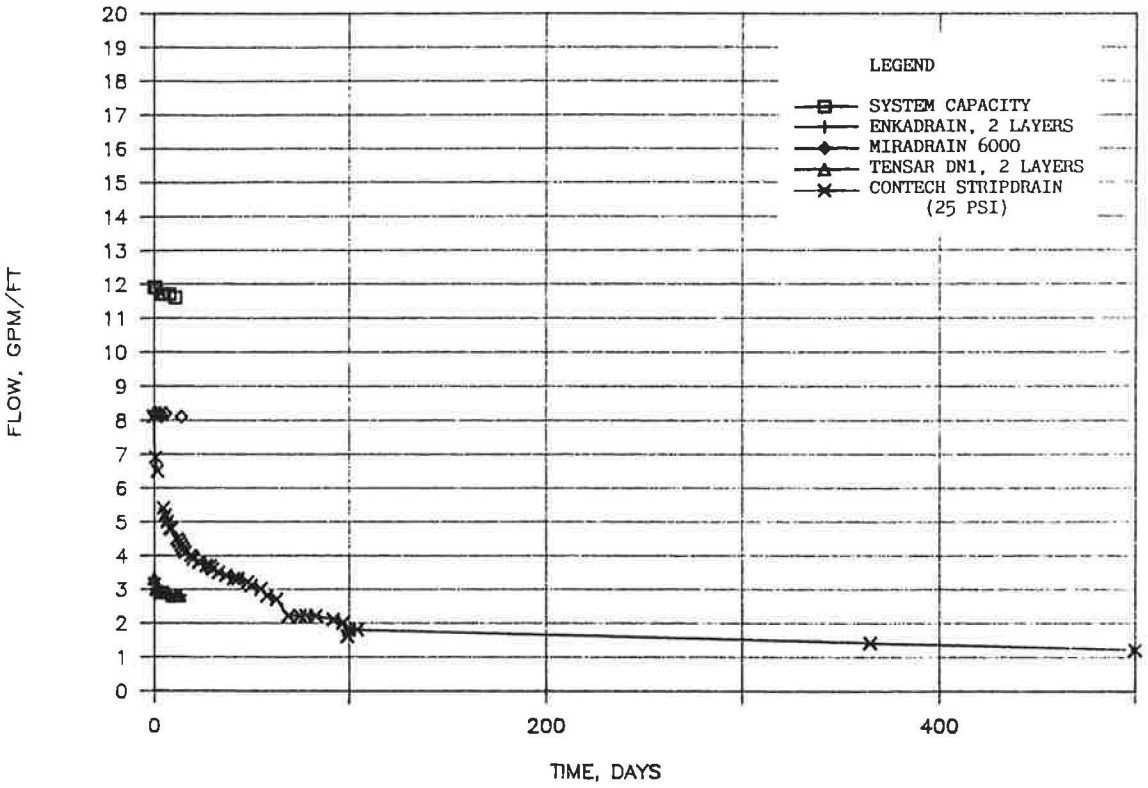

FIGURE 5 Equilibrium flow versus time (0–500 days, 30-psi confining pressure, 1.0 gradient): system capacity; Enkadrain, two layers; Miradrain 6000; Tensar DN1, two layers; Contech Stripdrain (25 psi).

future monitoring. These installations were placed behind fills or retaining walls. Instrumentation consisted of piezometers placed upslope and downslope of the drains.

Two of the installations were behind retaining walls on Stump Springs Road in the Sierra National Forest in central Sierra Nevada, California. This road, which is underlain by granitic bedrock, suffered extensive landslide damage in the spring of 1982 and the spring of 1983. Geocomposite drain systems were installed behind the Hilfiker welded wire retaining walls, which were constructed to repair the landslide damage. The geocomposite drain system used was the Eljen drain, which has a core similar to the Miradrain 4000. Construction of the walls and drain systems was completed in 1984.

The sites selected for future monitoring were Site 11, which involved a retaining wall approximately 150 ft long and 21 ft high at its highest point, and Site 17, which required a wall 100 ft long and 22.5 ft high. Installation of the drain system proceeded without difficulty. The rolls of Eljen drain came in lengths specified to fit the project conditions. As shown in Figure 6, the drains were pinned at the top and unrolled into the collection trench at the base of the drain system. With the drain system in place, the wall could be built easily. The completed wall at Site 17 is shown in Figure 7.

The monitoring equipment consisted of hand-fabricated piezometer sensors installed in front of and behind the drain system. The Omnidata Datapod Model DP212 was used to

TABLE 1 PERFORMANCE OF GEOCOMPOSITE DRAIN SYSTEMS AT CONFINING PRESSURE OF 30 PSI

Year Tested	Product	Equilibrium Flow, gpm/ft Gradient: 0.3	1.0	2.0
1986	System Capacity	6.8	11.7	16.9
1987	Amerdrain	3.0	5.2	7.5
1986	Contech Stripdrain*	0.6	1.2	1.8
1986	Enkadrain 9120, 1 Layer	0.4	0.9	1.2
1986	Enkadrain 9120, 2 Layer	1.5	2.8	4.1
1987	Miradrain 4000	0.4	0.9	1.2
1987	Miradrain 6000	4.6	8.1	11.8
1987	Tensar DN1, 1 Layer	0.8	1.5	2.2
1987	Tensar DN1, 2 Layer	1.6	2.8	4.0

* 25 psi confining pressure

FIGURE 6 Installation of drain system at Site 17 on Stump Springs Road.

scan the groundwater level detected by the sensors every 30 min and to record the average, high, and low readings every 24 hr. In addition, Site 17 had a high-capacity tipping-bucket rain gauge to monitor the drain system outflow. The outflow was recorded using an Enmos event recorder.

The instrumentation, monitored over a 4-year period, showed that the drain systems did perform as designed. As shown in Figure 8, the piezometers installed downslope of the drain system showed little or no groundwater, whereas the piezometers installed upslope of the drain showed fluctuating higher groundwater levels. Monitoring at Site 17 also showed a direct correlation between groundwater level behind the drain and outflow of the drain. The February 23, 1986, storm shown in Figure 8 was one of the largest in the area in the past 10 years, proving that this system can accommodate transient as well as steady-state groundwater conditions.

The third site monitored was on Mosquito Ridge Road in the Tahoe National Forest in northern Sierra Nevada, California. The project involved replacing a 25-ft-high fill damaged by storms in the spring of 1982. The site is underlain by slate bedrock. The geocomposite drain system installed was a double layer of Enkadrain 9120 placed along the bottom 6 ft of the fill. The two layers of Enkadrain, which comes with a geotextile glued to one side, were placed such that the geotextile side of one layer was in contact with the nongeotextile side of the other layer. A layer of Mirafi 700X woven geotextile completed this installation (see Figure 9). Once the drain system was installed, construction of the fill proceeded without difficulty. The project was completed in 1983 (see Figure 10). Instrumentation to monitor the groundwater levels at this site consisted of manually operated pneumatic piezometers. Monitoring of this site has revealed very little flow from the drain outlet, and no pore pressure buildup has been detected.

FIGURE 7 Completed wall at Site 17 on Stump Springs Road.

Other field installations of geocomposite drain systems without instrumentation involved systems fabricated at the site. These drains, which consisted of two layers of Tensar DN1 drainage net surrounded by a nonwoven geotextile, were placed behind geogrid-reinforced fills. These systems proved easy to fabricate and install.

Subdrains were also constructed along the edge of roads by trenching to a depth of up to 6 ft, installing the drain system (i.e., the Eljen drain or the Miradrain 4000), and backfilling the trench. All systems constructed to date have included perforated and nonperforated pipe networks to carry collected water.

CONCLUSIONS

On the basis of results of the laboratory work and field installations, the following conclusions were drawn:

- The flow capacity of geocomposite drain systems is directly proportional to the hydraulic gradient and inversely proportional to the confining pressure. These relationships can be determined using the test apparatus and method used in this study.
- The equilibrium flow of a product at a confining pressure can usually be determined after a period of several days. However, the confining pressure may need to be maintained for longer periods, up to several months.
- Reduction in flow capacity of the drain system may result from crushing or compression of the core material or from elongation of the geotextile due to increased soil pressures.
- All products tested, except for the Contech Stripdrain (which partially collapsed at 25 psi), had minimum equilibrium flow rates of about 1 gal/min per foot of drain width when subjected to a confining pressure of 30 psi and a hydraulic gradient of 1.0. The Contech product transmitted this volume at a confining pressure of 25 psi.
- Monitoring of several field installations of geocomposite drain systems has shown that these systems function as designed. As demonstrated during a severe storm, geocomposite drain systems can accommodate transient as well as steady-state groundwater conditions.
- All field installations were easy to install.
- For most Forest Service applications involving lateral pressures less than 15 psi and low flow rates, any of the tested systems would perform satisfactorily.

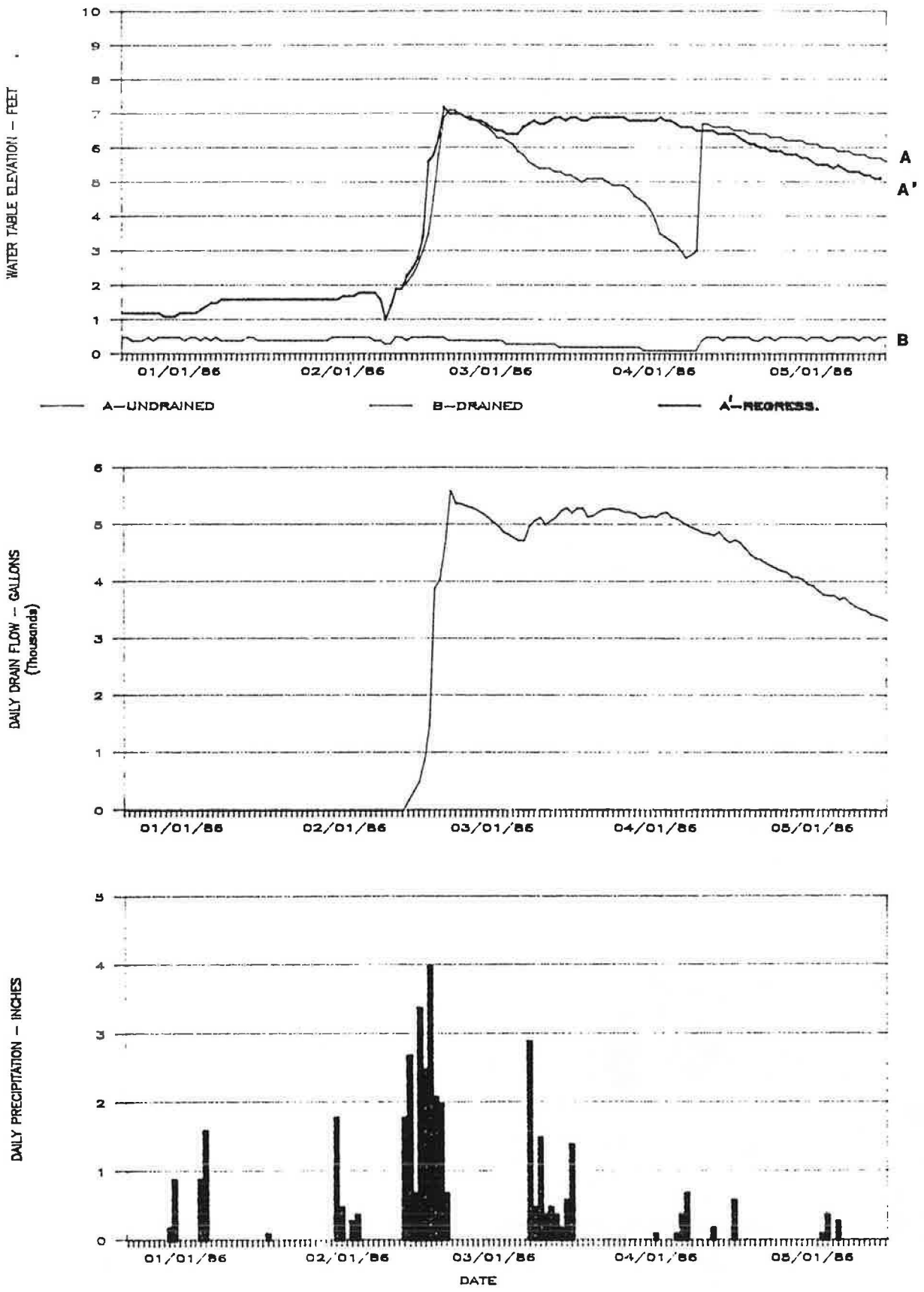

FIGURE 8 Site 17 on Stump Springs Road: (*top*) daily average water table elevation, (*middle*) daily drain flow, (*bottom*) daily precipitation.

FIGURE 9 Installation of drain system on Mosquito Ridge Road.

FIGURE 10 Completed fill on Mosquito Ridge Road.

ACKNOWLEDGMENTS

The authors wish to extend their appreciation to Rodney W. Prellwitz of the Forest Service's Intermountain Station for developing the recording groundwater monitoring equipment and methodology and to Alan Weaver and Pauline Cahill, formerly of the Sierra National Forest, for their assistance in monitoring the Stump Springs Road sites. Thanks are also extended to Lenny Lethaby and Phil Feldman of the Forest Service's Pleasant Hill Laboratory for their assistance in conducting the laboratory testing program.

Exploration, Design, and Construction of Horizontal Drain Systems

MICHAEL T. LONG

Horizontal drain systems for landslide correction have historically been installed with varying degrees of success. This variability of success has also been apparent in preconstruction exploration, engineering design, and construction and postconstruction monitoring. The USDA Forest Service, Region 6 (Pacific Northwest Region), has accomplished a number of successful horizontal drain projects in the past 5 years. As a result, a system-and-method approach has been developed, along with several low-cost alternative technology tools, for completing the work. This paper is not intended to be an inclusive discussion of all methods, nor an in-depth summary of quantitative detail, but rather a summary and guide to the project approach and a supplement to the current body of literature on horizontal drain systems.

Horizontal drains have proven to be a cost-effective alternative to major slope stabilization repairs, such as unloading and buttressing, when subsurface water is involved in the mechanics of failure. The practice of installing horizontal drains into unstable slopes to lower the phreatic surface or to relieve confined groundwater pressures has been in use for some time (*1*). The California Department of Transportation (Caltrans) pioneered drilled installations in 1939 (*1*). Since then, California has installed over 1 million linear feet of horizontal drains, helping to develop current state-of-the-art horizontal drilling and installation methods (*2*). The list of successful case histories throughout the United States is extensive. Material and site conditions in these cases have varied from discontinuities in overconsolidated clays to silty sands with rock fragments larger than 1 cubic yard (*1–8*). Brawner (*9*) has documented cases in which an induced vacuum system has improved performance of horizontal drains in rock slope applications

Although the literature contains many case histories on horizontal drains, the number of papers relating to preconstruction investigative techniques and quantitative design methods, especially in anisotropic heterogeneous material, is lacking. In a 1984 case history, Long (*7*) discussed such methods on one project. During the past 5 years, the USDA Forest Service in Region 6 has accomplished a number of successful horizontal drain projects using contract as well as force account drill crews. As a result of these successes, methods and procedures for accomplishing the investigation and design of these types of projects have been refined. The project planning process now includes a variety of techniques, including geophysical, geochemical, and drilling exploration methods; groundwater and drain system modeling using Darcy's Law and Mannings' Equation; and a method of drain end spacing developed by Prellwitz (*10*). A number of low-technology, inexpensive, and reliable exploration and installation techniques have also been developed (*11–14*).

These methods and procedures, as well as a summary of several case histories from the Forest Service in Region 6 and a 1958 case history from the Oregon Department of Transportation (DOT), are discussed in the following paragraphs. The groundwater modeling results obtained using the outlined approach are not intended to be precise; however, back calculations from postconstruction records of drain system discharge, groundwater levels, and rainfall have established a close correlation between initial design parameters and final system discharge.

PRECONSTRUCTION INVESTIGATION TECHNIQUES

The success or failure of any geotechnical project depends on the degree of accuracy of the subsurface model (soil and rock characteristics and horizontal and vertical distributions) and on the groundwater regime (distribution, volume, and flow characteristics). The most sophisticated analysis and design efforts are useless, within reasonable economic limits (regarding overconservative design parameters), unless an equal effort is employed toward technical confidence in the subsurface investigation phase. The following techniques can be considered for any geotechnical exploration effort but especially in drainage design for slope stabilization. Depending on the project scope and budget, some methods may not be economical but are presented as a suggested list of available techniques:

Area Reconnaissance

Area reconnaissance includes a complete literature search and aerial photo review as well as a general field reconnaissance to determine the history, process, and origin relating to the previous geologic and construction events that produced the present morphology. In particular, rock and soil units should be designated and classified, and point sources for groundwater infiltration should be identified.

Ground Control Survey

A ground control survey is best accomplished by a survey crew using an electronic distance measuring (EDM) device

USDA Forest Service, Region 6, Willamette National Forest, 211 E. 7th Ave., P.O. Box 10607, Eugene, Oreg. 97440.

for precision accuracy in control-point or hub-line monitoring of surface movements. The centerline and lateral cross sections should be surveyed under the supervision of a qualified engineering geologist or geotechnical engineer who can identify features relative to interpretation and stability analysis. These will be staked on the ground for future reference and included on the plan map and as cross-section points. Aerial photography targets can be set and tied in if the scope of the project warrants photogrammetric mapping. A topographic map and an adequate number of cross sections for analysis and design should be generated from the survey. Periodic control-point and hub-line resurveys should then be planned.

A lower-accuracy-level survey may be accomplished using the field developed cross section method (*11*), developed in Region 6 for internal geotechnical project support. This method uses a cloth tape, handheld compass, and clinometer and gives the investigator freedom of scale and the ability to operate independently of a survey crew. One limitation of this method is compounded errors in cross sections over 200 ft long. This method is ideal for small organizations without survey support, or with modest budgets, who need to begin or complete a project without delay.

Subsurface Interpretation

Before further exploration efforts, an initial approximation of the subsurface material distribution and slope failure geometry should be made on the centerline and lateral cross sections. This procedure compels the engineering geologist or geotechnical engineer to use the scientific approach of multiple working hypotheses leading to commitment to a working model. This, in turn, promotes confirmation or revision of the hypothesized model. As exploration proceeds, subsurface interpretation facilitates preliminary slope stability analysis and helps define further exploration efforts. This method has proven valuable for learning interpretation as well as giving management personnel a tool for serial review.

Drive Probe Exploration

Another low-technology exploration device developed in Region 6 is the Portable Drive Probe Assembly (*12*). The probe assembly is inexpensive, lightweight, and retrievable. It consists of 4-ft sections of ½-in. threaded galvanized pipe that are advanced below the surface by an 11-lb sliding hammer free-falling 41 in. The lead section is closed-end, by means of a pipe plug, and perforated with 3/16-in. drilled holes so water levels can be checked as the pipe is advanced. Normally, blow counts are recorded at 6-in. intervals. An increase or decrease in blow counts indicates a change in density and shear strength, relative to the overlying or underlying soil profile. These data will assist in making an initial interpretation of the subsurface. Measuring for free water in the hole through the pipe using a resistivity meter will also allow the investigator to determine exactly where groundwater was encountered. When apparent refusal is reached, additional dynamic force (accomplished by physically adding acceleration to the hammer) may be applied to ensure that a rock fragment has not been encountered. The pipe may be left in place as an open stand pipe piezometer, which will provide an extension to the data obtained from more costly efforts of core drilling. This device has been used effectively to depths of 30 ft.

Electrical Resistivity Profiling

A portable electrical resistivity instrument with a simple Werner configuration can extend the exploration efforts and help define areas of subsurface water concentrations. The profiles can also be used as an inexpensive way to plan drilling exploration for optimum placement of boreholes to intercept saturated zones. Profile lines with electrode spacings of 60 ft, recording drops in apparent resistivity 25 ft below the surface, have been used effectively.

Drilling Exploration

Hollow-stem augers and continuous standard penetration test sampling should be used to obtain subsurface samples and soil strength estimates without introducing drilling fluids into the borehole. The hole should be advanced far enough beyond the interpreted failure plane, using a core barrel assembly if necessary, to properly seat and seal any borehole instrumentation such as inclinometer casing or piezometers. At a minimum, open stand pipe observation wells should be installed. Observation wells must be installed within the failed mass at points near or adjacent to analysis cross sections to determine the effectiveness of the horizontal drains and the final calculated factor of safety (F.S.) achieved through reduction in pore water pressure. Additional borings and wells are suggested beyond the lateral failure limits to fully define the groundwater model and to obtain data for constructing lateral cross-section end areas. These additional borings and data will also facilitate horizontal flow net construction to determine the most effective drain locations. As the borings and observation well installations are being completed, the design must be considered relative to in situ permeability testing. Slug tests, or maintained head tests, have proven to be efficient. This testing should be completed to obtain the coefficients of permeability for hydraulic analysis. The up- and downslope boreholes and measured static water levels should be plotted on the appropriate cross sections to determine the hydraulic gradient for the model. All drilling exploration should be completed with a qualified engineering geologist or geotechnical engineer on-site as inspector.

Permeability Testing

In situ permeability tests should be performed in the material to be drained (*12*). If, from the exploration borings, the material is determined to be hydraulically zoned, containing perched water tables or pockets of isolated water, testing in an adjacent borehole is preferable so each zone can be tested separately. Testing can be accomplished in the observation wells if care is taken in placement and sealing of the wells within the zone to be tested, or pneumatic borehole packers can be used.

Groundwater Tracing

There are several methods used to confirm a point source or sources of water infiltration into a system and to determine hydraulic conductivity between observation wells. The tracer dyes rhodamine WT and fluorescein have been used successfully in groundwater modeling efforts by the Forest Service in Region 6 (*7,15–17*). Both dyes can be used concurrently to determine separate point sources at the surface (which could discharge at the same location) or introduced into observation wells. Water samples can be collected directly or by placing a packet of activated charcoal at discharge points in the failed mass. Charcoal packets can also be connected to a line lowered into a borehole. Sodium chloride has been used to trace groundwater and to determine hydraulic conductivity by measuring the relative electrical conductivity at the discharge point (*18*). However, this method is limited to shorter travel distances because of the lower concentrations of sodium chloride achievable and detectable. A weir should be constructed at all springs and seeps observed at the slide scarp, lateral margins, and toe. This can be accomplished using natural material and fitting a 1-in. polyvinyl chloride (PVC) overflow pipe through the weir so flow measurements can be taken of all the seeps and then summed for water budget estimates. The weir discharge must be directed away from the failed mass.

Water Surface Contours

A general water surface contour map should be constructed from the static water level (SWL) readings, converted to elevations, from the observation wells. The water surface contours can then be superimposed on the topographic contours and used to plan the drainage design. The water surface contours can generally be considered equipotential lines, and the general subsurface flow path or paths can be estimated by constructing flow lines perpendicular to the equipotential lines.

Test Drain Installation

Test drains should be installed to confirm final drain locations. The test drains should be located according to the water surface contour model in areas that suggest piezometric valleys (concentrations or convergence of flow lines).

DRAINAGE SYSTEM DESIGN

In order to design a horizontal drainage system, the data obtained from the subsurface investigation and subsequent interpretation must be used to obtain the variable values necessary for Darcy's Law, Mannings' Equation, and drain end spacing equation parameters. The following is a recommended approach to the design (the number of drains needed, inclination, length, and maximum end spacing).

Groundwater Recharge Capacity

To determine the volume of water entering the failure area, which ideally corresponds to the desired interception volume, Darcy's Law states

$$Q = kia \tag{1}$$

where

Q = discharge (gal/min),
k = coefficient of permeability (ft/day),
i = hydraulic gradient (10^{-2}), and
a = cross-sectional area (ft^2).

The cross-sectional area is determined from the interpreted cross sections perpendicular to the long axis of the slide, taking into consideration the current (steady-state flow) and potential (transient-state flow) rainfall recharge areas. The hydraulic gradient can be obtained from the difference in head and horizontal distance between upslope and downslope observation wells. Coefficients of permeability are determined by borehole falling and maintained head tests performed in the exploration phase.

This process should be used for each water zone if possible, and summed for the total discharge. Current cross-sectional area discharge calculations should then be compared with the sum of all seep discharge points to determine if there is a reasonable correlation. If the correlation is not reasonable (e.g., the sum of all seeps does not represent the total discharge, or values assigned to the model parameters are high or low), engineering judgment or further investigation must be used to resolve the discrepancy.

Number of Drains Needed

The number of drains necessary to accommodate the computed volume is based on Mannings' Equation, which states

$$V = \frac{1.486}{N} R^{\frac{2}{3}} S^{\frac{1}{2}} \tag{2}$$

where

V = velocity (ft/sec);
N = roughness coefficient = 0.009 (*7*);
S = percent slope, range 2 to 15 percent (*7*); and
R = hydraulic radius = area/wetted perimeter = 0.031 ft (for 1½-in. pipe flowing full).

The capacity of 1½-in. inside diameter (ID) slotted PVC pipe flowing full in various slope gradient configurations can then be calculated by

$$Q = VA \tag{3}$$

where

Q = discharge, and
A = end area of 1½-in. drain pipe (0.012 ft^2).

Drain grades and lengths are then determined from cross-section analysis to optimize the drain length in the saturated

zones. An array of grades and lengths will likely be necessary to accommodate each site-specific geometry.

On the basis of case history experience, it is suggested that the number of drains in the preliminary design be based on 25 percent flow capacity to compensate for nonfunctional or low-performing drains in the system. Consideration should be given to incorporating interceptor drains above the failed mass, if possible, as well as relief drains within the failed mass.

Slot Size

The slot size of the drain pipe must be small enough to prevent piping of fines through the opening but large enough to prevent clogging. FHWA (*19*) recommends that the slot size be equal to one-third of the D_{85} of the soil for slotted underdrain systems. Cedergren (*20*) suggests that

$$\frac{D_{85}\ \text{soil}}{\text{Slot width}} < 1.2 \tag{4}$$

where "D_{85} is defined as the decimal number of the size of soil particles for which 85 percent of the soil is finer. . . ." (*17*). On the basis of observation and performance, the Cedergren estimate has given values for slot sizes larger than desirable, which has produced piping. It is recommended that the FHWA value be used.

Maximum End Spacing

The minimum phreatic surface drawdown that will produce an increased shearing resistance for a desired F.S. is determined by the distance between drains in parallel installations. If the drains are installed in a fan array, the maximum end spacing should be calculated to ensure that the minimum required drawdown is being affected within the failure mass. Prellwitz (*10*) outlined a method for determining end spacing on the basis of a modification of the Hooghoudt Equation for transient-state flow and the site-specific slope and phreatic surface geometries.

Collector System

The collector system for a large array of drains may consist of several options. For example, 12- or 8-in. corrugated pipe anchored above ground with steel posts and ¼-in. wire rope, with 2-in. feeder hoses clamped to the drain ends, has been used successfully (see Figure 1). A buried pipe may be considered if further movement is not expected, which would make it difficult to locate a break.

Corrugated polyethylene pipe can be used in a subsurface installation. Surface installations should incorporate material that is strong enough to withstand animal traffic or vandalism. If freezing conditions are likely at the site, a covered manifold system with a concrete drop collection box at the drain discharge points can be designed. Access to each individual drain must be provided to allow monitoring of postconstruction discharge and to facilitate cleanout. A sudden decrease or increase in flow, or a change in water color, may indicate further movement.

FIGURE 1 Collector system intake point.

STANDARD CONSTRUCTION

Rotary drilling is the common method of horizontal drain construction. Installation is accomplished by advancing a 4-in. drill casing to the desired length using a knock-off tri-cone roller bit. At the end point, the casing is rotated in the reverse direction, and PVC pipe slotted in two rows on 120-degree centers is inserted through the casing, thereby knocking off the bit. As the casing is removed, the PVC pipe and roller bit remain in the drill hole.

Suggested Construction Practices

Suggested construction practices include the following:

- The collector system should be constructed before drilling to accommodate anticipated drain flow.
- If excavation is necessary to construct drilling pads, steps must be taken to ensure that the drill pad cutslope will continue to be stable under leaky drainage conditions. If necessary, a rock buttress should be designed and constructed for local stability before drilling. If the cutslope fails after construction, the drains can be sheared off and cause system failure.
- If possible, drilling should take place during wet weather to allow field judgments to be made for modifications on drain locations, concentrations, direction, and inclination on the basis of observed discharge from completed drains.
- Drain end elevation should be determined using the manometer method (a hose connected to the end of a flowing drain, elevated to equilibrium) or a pressure meter to check for up or down casing drift. The next hole should then be corrected as necessary.
- Absorbent wipes, stream booms, silt fences, and straw bales are effective in controlling sediment from drill cuttings and machine fluid leaks. If adjacent to an environmentally sensitive area, a spill plan may be necessary.

- Slots have previously been installed in both the up and down positions successfully; however, consideration must be given to segments of blank pipe in the drain. Blanks should be installed in any segment that is not penetrating the water-bearing zone to prevent migration of groundwater into otherwise dry areas. The blanks should be installed in at least the last 20 ft of drain if toe or cutslope stability is a concern. Drains should not penetrate further than 15 ft beyond the failure surface or the drainage barrier.
- For discharge end protection, and to prevent root growth in the drain, a 3-in. galvanized metal pipe should be installed over the discharge end into the drill hole to a minimum of 5 ft, then grouted in place.

Inspector Duties

A qualified engineering geologist or geotechnical engineer should direct the drilling installation to ensure that design criteria (such as angle, elevation, and location) are met and to make any field modifications. Routine duties should include

- Setting fore and aft site stakes for hole alignment before any large metal objects arrive that would affect compass bearings;
- Measuring the drill casing slope as the hole is collared in;
- Recording advance rate, water return, and water color;
- Monitoring the path of the drill casing for surface indications of drilling fluids in any adjacent or upslope tension cracks;
- Noting material changes with casing advancement;
- Sampling the drill cuttings to determine when the failure surface or the soil and rock interface has been reached;
- Having on-hand predictive tools (such as interpreted cross sections and drill logs) to assist with estimating failure plane and material boundaries;
- Recording final length, slope, end elevation, and discharge rate for each completed drain; and
- Ensuring that the drain number is marked on the galvanized sleeve with a metal stamp for future reference.

ALTERNATIVE CONSTRUCTION METHOD

The State of Oregon, Department of Forestry, has experimented with driven horizontal well points as an alternative low-technology method of installing horizontal drains into slopes. The technique uses sections of ½-in. steel pipe inserted into a perforated 1-in. PVC pilot sleeve with a hard plastic well point. The well point is then advanced into the slope by means of a slide hammer acting against the ½-in. steel pipe (similar to the drive probe method). Sections of PVC and drive pipe are added as the drain is advanced. The drive pipe is then rotated and removed from the PVC upon completion. This system has been used effectively with horizontal advancement up to 40 ft. With some modification, it could be used with a head frame assembly and power cathead to drive steel well points and galvanized pipe to even greater depths with faster penetration rates.

POSTCONSTRUCTION MONITORING

Postconstruction monitoring of a drainage system should be calculated into the project budget and scheduling. Monitoring should continue on a weekly basis for the first month after the project and monthly thereafter until there is a high degree of confidence regarding long-term stability. Monitoring frequency may be increased during periods of excessive rain, rain-on-snow events, or spring runoff. The following is a list of recommended monitoring activities:

- Measuring static water levels in all observation wells,
- Obtaining inclinometer readings,
- Resurveying hub lines and control points,
- Measuring the discharge from each individual drain,
- Measuring total system discharge, and
- Installing a rain gauge on site and recording precipitation.

All water levels, total system discharge, and rainfall can be recorded automatically using pressure transducers calibrated to the head of water in a casing, flume, or rain gauge and connected to a battery-powered automated data logger. Prellwitz (*14*) provides detailed instructions on construction of these devices.

A low-technology device for recording the highest water level reading in an observation well is to place finely ground cork in a length of ⅜-in. clear flexible-plastic tubing (equal to the depth of the borehole) with a piece of sponge to close the bottom end. The riser tube is then lowered to the bottom of the observation well and fastened in place with tape. As the water rises in the observation well, the cork in the plastic tubing rises to the highest level in the riser tube. As the water level decreases, the cork adheres to the sides. The tubing can then be removed from the observation well at any time, with the highest level of cork representing the highest level of water that has been in the well. The cork can then be flushed back to the bottom of the riser tube, and the tubing reinserted in the well.

Drain effectiveness is often reduced within 5 to 10 years due to root growth, piping of fines, and bacteria. Caltrans (*2*) recommends that an ongoing inspection program be initiated, and that a cleaning schedule be established if reduced discharge volume is noted. Cleaning is accomplished using a high-pressure water pump with a self-propelling jet nozzle attached to a length of hose inserted the full length of each drain.

CASE HISTORY SUMMARIES

The five case histories summarized below represent between $300,000 and $400,000 cost savings over the next lowest cost stabilization alternatives considered.

Camp 5 Slide

Location:	Willamette National Forest, Oakridge, Oregon
Failure mass:	250,000 yd³
Install dates:	December 1983 to January 1984
Linear feet:	7,800 ft

No. drains:	52
No. locations:	7
Drain length:	65 to 240 ft
Drain slope:	2 to 15 percent
Slot size:	0.050 in.
Soil type:	Silty sand (SM)
SWL drop:	14 ft
Total discharge:	High = 576 gal/min
Install cost:	$107,000
Final F.S.:	1.20
Investigation:	
Drill holes:	21
Drive probe:	0
Other:	Resistivity profiling, dye tracing, permeability testing, EDM survey, aerial photogrammetry
Comments:	At least three previous attempts to stabilize the slide were unsuccessful (unloading and buttressing). The slide increased in size from 30,000 to 250,000 yd^3 in the final failure before drain installation.

Fairview Sanitary Landfill

Location:	Bureau of Land Management, Coquille, Oregon
Failure Mass:	80,000 yd^3
Install dates:	April 1987 to May 1987
Linear feet:	3,337 ft
No. drains:	19
No. locations:	3
Drain length:	175 ft
Drain slope:	3 degrees
Slot size:	0.051 in.
Soil type:	Sandy silt (ML)
SWL drop:	5 ft
Total discharge:	10 gal/min
Install cost:	$45,000
Final F.S.:	1.25
Investigation:	
Drill holes:	9
Drive probe:	0
Other:	Survey
Comments:	Drains were placed to intercept subsurface water in siltstone. Drains were also placed into landfill pits to drain infiltrated water. The majority of the flow was from intercepted subsurface water. An immediate maintenance concern arose because of buildup of iron bacteria in the drains.

Powder Creek Slide

Location:	Willamette National Forest, Oakridge, Oregon
Failure mass:	55,500 yd^3
Install dates:	August 1988 to October 1988
Linear feet:	2,754 ft
No. drains:	20
No. locations:	1
Drain length:	110 to 225 ft
Drain slope:	3 to 10 percent
Slot size:	0.090 in.
Soil type:	Silty sand (SM)
SWL drop:	10 ft
Total discharge:	4 to 16 gal/min
Install costs:	$30,000
Final F.S.:	1.35
Investigation:	
Drill holes:	22
Drive probe:	2
Other:	Survey
Comments:	Drill pad construction at the toe of the slide was difficult due to saturated conditions. The drains have effectively stabilized the road prism, which has been moving for 20 years despite previous attempts at stabilization (piles, relocation, syphon wells). The drill pad backslope was not buttressed, and drain slots were installed the entire length. As a result, the pad backslope failed and sheared all drains 15 ft behind the discharge point. The drains are still effective; however, the toe will have to be restabilized.

Quentin Slide

Location:	Willamette National Forest, Blue River, Oregon
Failure mass:	66,600 yd^3
Install dates:	February 1987 to March 1987
Linear feet:	4,087 ft
No. drains:	19
No. locations:	2
Drain length:	215 to 296 ft
Drain slope:	2 to 14 percent
Slot size:	0.050 in.
Soil type:	Silty sand (SM)
SWL drop:	10 ft
Total discharge:	High = 40 gal/min
Install cost:	$60,000
Final F.S.:	1.05
Investigation:	
Drill holes:	13
Drive probe:	10
Other:	In situ permeability testing
Comments:	The system has not been monitored since installation. To obtain the original road alignment, further stabilization methods need to be considered.

Highlands Interchange Slide

Location:	Sunset Highway, Portland, Oregon, Oregon DOT
Failure mass:	300,000 yd^3
Install dates:	October 1958 to November 1958
Linear feet:	5,900 ft
No. drains:	22
No. locations:	7
Drain length:	80 to 450 ft
Drain slope:	3 to 5 percent
Slot size:	½-in. drilled holes
Soil type:	Sandy silt (MH)
SWL drop:	30 ft
Total discharge:	100 gal/min
Install costs:	Unknown
Final F.S.:	Unknown
Investigation:	
Drill holes:	17
Drive probe:	None
Other:	Unknown
Comments:	This system was installed in 1958 using 2-in. iron pipe with drilled holes to stabilize a slope failure in a residential neighborhood caused by highway widening. The drains are still operational today after 31 years of service. Maximum discharge from one drain is still 15 gal/min.

CONCLUSIONS

Horizontal drains are a cost-effective alternative to slope stabilization when elevated pore water pressures must be reduced. The chances of success for any geotechnical project depend as much on the quality of exploration and interpretation as on the design. This premise is even more important when groundwater variables are the focus. By applying a system-and-method approach with working hypotheses, the probability of success is greatly increased. Lower-cost technology exists and is being refined, which will allow an agency or organization with modest resources to successfully complete several or all phases of a project.

ACKNOWLEDGMENTS

The author would like to thank the many workers responsible for the cumulative level of technical expertise in the field of horizontal drain exploration, design, and installation. Special recognition is extended to the following individuals for information on specific topics: René Renteria of the Willamette National Forest for Fairview and Powder Creek slides, Richard Kennedy of the Willamette National Forest for Powder Creek Slide, Peter Bolander of the Willamette National Forest for Camp Five and Quentin slides, Donald Turner of the Oregon DOT for Highlands Interchange Slide, David Michael of the Oregon Department of Forestry for the driven well point method, Douglas Williamson (retired) of the Willamette National Forest for the drive probe method, Kenneth Neal of the Olympic National Forest for the Field Developed Cross Section, and Rodney Prellwitz of the Forest Service, Intermountain Research Station, for information on the ground cork and riser tube method of obtaining remote high-water levels.

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Liquid Calcium Chloride for Dust Control and Base Stabilization of Unpaved Road Systems

HENRY KIRCHNER AND JAMES A. GALL

The use of liquid calcium chloride on unpaved roads as a dust control agent and as a base stabilization material is examined. In the first section, a description is provided of how calcium chloride controls dust and the benefits it produces. The performance of calcium chloride is compared with that of other commonly used dust control agents. Recommended guidelines and application rates for controlling dust on unpaved roads with calcium chloride are then given. The second section provides a description of how calcium chloride stabilizes unpaved road bases and lists the seven ways the chemical helps build stronger roads. Recommended guidelines and application rates for stabilizing road bases with calcium chloride are provided.

Calcium chloride ($CaCl_2$) is a simple material produced from natural brine deposits found underground or from the synthetic Salvay process. It is processed into a colorless, odorless liquid, which is the material primarily used for dust control and base stabilization on unpaved roads. Calcium chloride is also processed into white flakes and white pellets. These products are occasionally used for dust control and base stabilization.

CALCIUM CHLORIDE AS A DUST CONTROL AGENT

Calcium chloride has two characteristics that enable it to be useful for dust control applications. First, it is hygroscopic. In other words, it attracts moisture from the atmosphere and surrounding environment and resists evaporation as it works to remain in its natural liquid state. Second, calcium chloride is deliquescent, which means the solid form can dissolve into a liquid by absorbing moisture from the atmosphere and surroundings. When calcium chloride is spread on low-volume unpaved roads in the spring, its moisture-attraction ability works to keep the surface damp and to keep dust down, usually throughout the summer.

Calcium chloride has other properties that contribute to the improvement and performance of unpaved roads. For example, compared with plain water, calcium chloride has a stronger moisture film, higher surface tension, lower vapor pressure, and lower freezing point. The combination of these properties enables the chemical to keep unpaved surfaces damp and to keep fines, or tiny dust particles, in place. Additionally, calcium chloride actually helps bind the aggregate particles together and, as a result, the surface becomes compacted by traffic. Over time, calcium chloride slowly penetrates the surface by several inches, which creates a stabilizing effect to the road. The longer calcium chloride is used, the more stability that is achieved. Finally, the chemical's lower freezing point helps unpaved roads resist frost heave in late fall and early winter.

Technical Service and Development, The Dow Chemical Company, 2020 WHDC, Midland, Mich. 48674.

Benefits

As a rule, one car making one pass on 1 mi of untreated, unpaved road everyday can generate 1 ton of dust in 1 year. When the road is treated with a dust suppressant, however, it retains a high percentage of the fines it would otherwise lose as dust. Road superintendents from the Midwest have verbally reported up to an 85 percent reduction in fines from road treatment.

Because calcium chloride is hygroscopic, it holds fines in place. Therefore, coarse aggregates tend to stay in place, eliminating their abrasive action. This reduces the need for aggregate replacement and, in some cases, can eliminate it for long periods of time, depending on the road's average daily traffic.

Because road materials stay in place, the frequency of blading can be reduced from 25 to 75 percent by using a dust palliative. This, in turn, can reduce labor and equipment costs.

Also, fewer spot repairs are needed, which means less labor and materials are required.

Finally, less dust, less aggregate replacement, and less blading mean less repair work in general. Therefore, less fuel is used and less equipment maintenance is needed.

Comparison with Other Dust Control Materials

Many different materials under many different brand names have been used for controlling dust, which has led to confusion regarding their performance. The following is a review of some of the more commonly used dust control agents and how they compare to calcium chloride.

Oil and Asphalt Emulsions

In a year-long study commissioned by Dow Chemical and conducted by PEI Associates, Inc., an independent research

firm located in Golden, Colorado, sections of a road in Adams County, Colorado, were treated with an asphalt emulsion and calcium chloride. Data gathered after only 4 weeks of testing indicated that liquid calcium chloride had already achieved a dust control rating efficiency 135 percent greater than the asphalt emulsion (see Table 1).

Due to the emulsion's poor performance, a second coating was applied after Week 4. Shortly thereafter, the emulsion-treated section was termed unsatisfactory and the emulsion was removed from testing.

The emulsion-treated section of road was visibly inferior compared with the calcium chloride–treated section. For example, while the calcium chloride section was firm, smooth, and practically dust free, the emulsion section was severely rutted. This condition resulted because emulsions offer little aggregate binding capabilities. Instead, they coat an unpaved surface with a thin crust that can fragment under loads and leave behind potholes. It was also observed that considerable aggregate had been lost alongside the road.

In a 1979 study, Harvard University (*1*) compared the use of oil emulsions and calcium chloride on unpaved roads. The study is considered to be one of the most comprehensive conducted on dust control agents. It concluded that treating unpaved roads with calcium chloride is far more economical than treating them with oil. This conclusion was reached by taking the average total annualized cost of various treatments for an unpaved road and dividing it by the dust control efficiency (dust emission reduction) for each method. Specifically, to determine the cost of unpaved road treatments, the study took into account the initial capital investment for each road, the capital recovery factor (percentage of initial investment that would be paid yearly on a loan at a certain interest rate for a specified number of years), and the average annual cost of operation and maintenance.

Because oils do not perform as well as calcium chloride, they would have to be applied more frequently to achieve a similar degree of dust control. This factor probably had a greater bearing on the difference in cost between the two materials. In gathering their data, the Harvard researchers requested cost estimates of numerous state highway departments during the 1970s. Figure 1, which reflects these data, shows the mid-range of these estimates. It demonstrates that treating unpaved roads with oil can be six times more costly than using calcium chloride.

A recent article (*2*) referred to oil products used for dust control and noted that,

> Generally, the life expectancy of dust palliatives decreases with higher traffic volumes and with higher percentages of truck traffic. This is particularly true of products that create a hard surface crust, which is subject to potholing, such as . . . most petroleum products.

TABLE 1 SUMMARY OF CONTROL EFFICIENCIES, WEEKLY AVERAGES

	Liquid Calcium Chloride (%)	Asphalt Emulsion (%)
Week 1	77.3	43.0
Week 2	69.5	25.8
Week 3	74.2	33.6
Week 4	70.8	25.8
Overall average	72.6	31.1

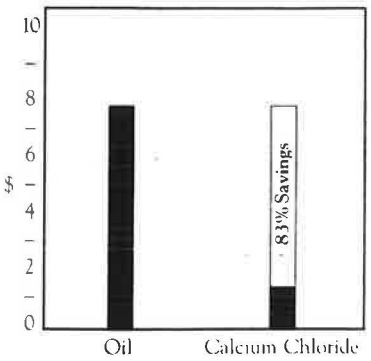

FIGURE 1 Cost comparison for unpaved road treatments.

In addition to higher costs and poorer performance, oils and emulsions have other problems. For example, because they coat an unpaved road and do not penetrate the surface, car tires can pull them up along with road material and leave behind potholes. They are dirty and messy, and they stick to cars, clothing, and shoes. They can be tracked into buildings, and they can choke roadside foliage. Further, after applying an oil or emulsion, a curing time is necessary before traffic can be allowed on the road again. These problems do not occur with calcium chloride.

Magnesium Chloride

Like calcium chloride, magnesium chloride in solution attracts moisture and resists evaporation to control dust on unpaved roads. However, at temperatures above 71°F and at relative humidities below 31 percent, magnesium chloride begins to lose these capabilities whereas calcium chloride does not (see Figure 2).

The costs of liquid magnesium chloride versus liquid calcium chloride are about equal (depending on shipping destinations), but only in terms of purchase price. According to application rates typically recommended by the suppliers of each chemical, about half as much calcium chloride is needed for dust control. Calcium chloride controls dust more effectively than magnesium chloride, particularly at high temperatures and low relative humidities. Therefore, on an application basis, calcium chloride could cost about half as much (see Table 2).

A similar finding was noted in a 1972 study conducted by The Royal Technical College of Stockholm, Sweden. There, researchers concluded that during typical summer conditions

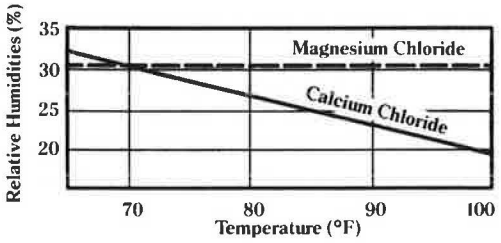

FIGURE 2 Temperature and humidity required to keep each chemical in solution.

TABLE 2 SOLUTION CONCENTRATION AND APPLICATION RATE RECOMMENDED BY SUPPLIERS

	Concentration (% by weight)	Gallons Spread (per yd^2)
Calcium chloride	38	0.27
Magnesium chloride	26–32	0.50

> It is necessary that about 18 percent more magnesium chloride commercial product be applied . . . to get the same dust control effect (as calcium chloride provides). With the present cost accounting situation the calcium chloride should come out as being cheaper to use than magnesium chloride, while achieving the same dust binding effect.

Furthermore, magnesium chloride is generally produced by solar evaporation. The more exposure to the sun it gets, the higher the chemical concentration will be; the less exposure, the lower the concentration. Because this production method depends on the weather, magnesium chloride concentrations and performance can vary widely.

Conversely, calcium chloride is a processed material under strict quality-control conditions manufactured according to ASTM standards. Chemical concentrations may vary only slightly from batch to batch, which in turn means consistent performance.

Lignin Sulfonate

In 1983, the U.S. Bureau of Mines (*3*) conducted field tests on three mine haul roads located in various parts of the country. The objective was to determine the cost effectiveness of achieving a minimum 50 percent level of dust control on these roads. It was found that lignin sulfonate, a byproduct of paper mills, costs between 1.41 and 1.46 times more than liquid calcium chloride to achieve the 50 percent level of control efficiency required.

Lignin sulfonate, like petroleum-based dust suppressants, provides little soil penetration. Surface coating of aggregate is the means of dust control.

Water and Surfactant

The U.S. Bureau of Mines (*3*) also tested plain water and found it to cost between 1.48 and 2.18 times more than liquid calcium chloride to achieve the 50 percent level of control efficiency required.

The surfactant, which is a soap-like material, was removed from consideration because it did not allow for an estimation of required application frequency to achieve a 50 percent level of dust control.

Recommended Guidelines and Application Rates

Unpaved roads with an average daily traffic (ADT) of 25 usually do not need dust control. However, they should be evaluated on an individual basis, particularly with respect to safety or health considerations. For example, dust control may be needed on a road where the nearby residents suffer from respiratory problems.

Roads with an ADT between 50 and 100 can be spot treated for dust control, particularly at intersections and railway crossings; in front of schools, hospitals, and residences; and near dust-sensitive crops. On roads with an ADT of 100 to 500, the greater loss of road material will probably justify treating either the center strip or the full width. Roads with an ADT greater than 500 can be considered for paving. However, dust control measures can help stabilize the surface until funds are available.

Many dust suppressants do not work well in sandy, nonplastic soils. Sandy soils are porous and lack the cohesiveness to retain the suppressant. In other words, the dust control agent tends to migrate through this type of soil.

Also, these suppressants do not work well in soils containing greater than 25 percent clay because they are hygroscopic. When the materials are combined, they are likely to attract too much moisture to the clay and, therefore, tend to make an unpaved road too damp or wet. This condition may result in loss of road stability to support vehicle traffic.

Before treating an unpaved road with calcium chloride a soil sample should be taken and analyzed. If the road has too much sand or clay, an amount of the appropriate material should be added to compensate for any deficiencies. A typical wearing course is presented in Table 3.

The decision to use liquid or dry calcium chloride is usually based on economic considerations and the type of storage, mixing, and application equipment available. Liquid calcium chloride is recommended because it can be more evenly distributed on the road. For dust control, liquid calcium chloride is usually purchased in 30 to 42 percent solutions.

Alternatively, users can sparge (mix with water) flake or pellet calcium chloride products on location to produce a liquid. Flake or pellet calcium chloride can also be spread directly onto unpaved surfaces without first being put into solution. Special consideration must be taken to make sure adequate moisture is available for the dry calcium chloride. Water is usually added to the unpaved surface before or after the calcium chloride has been applied.

Because calcium chloride works by attracting moisture from the atmosphere and surroundings, it is best to apply the chemical after seasonal spring rains when there is ample moisture in the ground. Applications should not be started during a heavy rainfall or if rain is threatening. If it is a dry spring, water should be sprayed on the road before application, provided the moisture can soak into the ground and will not run off.

TABLE 3 AVERAGE GRADATION FOR A TYPICAL WEARING COURSE

Sieve Designation	Percent Passing
1 in.	100
3/4 in.	85–100
3/8 in.	65–100
#4	55–85
#10	40–70
#40	25–45
#200	10–25

Calcium chloride typically retains its moisture-attraction ability throughout the summer because it resists evaporation. Therefore, it should continue to control dust throughout this period as well. A second application is recommended in late summer or early fall to provide dust control into late fall and to help protect unpaved roads against early frost damage.

The following three steps are needed to provide effective dust control with calcium chloride.

Step 1. The road surface should be bladed to a depth sufficient to remove potholes, washboarding, and ruts, then shaped to a straight-line slope of ½ to 12 in.—a type "A" crown. On curves, the slope should remain the same across the entire width of the road (1½ to 12 in.). The transition between a straight line and curve should be gradual. This shaping lets water drain off the road, which in turn helps prevent the formation of potholes. If water is allowed to stand on a road surface, it can act as a particle lubricant and create soft spots. As cars pass over these soft spots, tires can push the soil and aggregate aside and into ditches.

Any soil dams, or berms, that are created during blading should be removed. Otherwise, these buildups can restrict water drainage from the road surface. Berms can be removed by blading them smooth along with the road's surface as part of the final touch-up in the road-shaping process.

Step 2. A 38 percent solution of liquid calcium chloride should then be applied to the road surface at the rate of 0.27 gal/yd^2. Experience has shown that this percentage is the ideal calcium chloride concentration for dust control. If a weaker solution were used, it would lose its dust control effectiveness in a relatively short period of time and the road would have to be retreated. A stronger calcium chloride solution would tend to bead up on the road during application rather than penetrate the surface.

Alternatively, 1.54 lb/yd^2 of flake calcium chloride or 1.32 lb/yd^2 of pellet calcium chloride can be applied to achieve effective dust control.

Step 3. As previously mentioned, dust control is usually maintained throughout the summer with minimal attention. However, a second treatment is recommended in late summer or early fall. It may be necessary to reblade the road according to Step 1 before applying calcium chloride.

CALCIUM CHLORIDE AS A BASE STABILIZATION MATERIAL

Soil stabilization is a means of upgrading the engineering properties of soils to provide maximum return on investment in road construction or improvement. Although there is no precise definition of stabilization, a soil is said to be stable when it resists change, particularly mechanical change, over long periods of time. Conversely, an unstable soil is one that breaks up, shifts, or sinks when acted upon by the normal forces of load and climate, resulting in a prematurely deteriorating surface.

Three mechanical aspects are critical to a soil's ability to withstand loads: (a) cohesion, (b) friction, and (c) density. Cohesion refers to the ability of soil particles to stick together. Friction refers to the ability of particles to resist shifting of their position relative to each other. Density is the weight of a material to its bulk.

Damp clay has a high density and good cohesiveness. However, its low coefficient of friction permits its particles to shift along several axes and, under load, it yields easily. Gravel has a high coefficient of friction, somewhat lower density, and little or no cohesiveness. Although gravel can bear large static loads, it is also porous and therefore extremely unstable and susceptible to uneven settling, hydraulic action, and frost heave.

Because no material exhibits both a high coefficient of friction and high cohesiveness, a mixture of materials is required. In fact, several aggregate grades should be involved, ranging from coarse aggregate to fines passing a No. 200 sieve. Figure 3 shows the average gradation of a stabilized mix on a typical base course.

When all of these materials are mixed together, the larger components will leave voids, which are filled in by progressively smaller components. A chemical additive, such as calcium chloride, then coats the various-sized particles and adds density to the mix, which helps prevent their loss due to traffic or weather. This process is referred to as soil stabilization.

Compacting a subgrade is important because it can increase bearing capacity by as much as 150 percent, increase shear strength, reduce permeability, and improve overall stability. The result is a stronger, more durable road whose bituminous or concrete surface can be thinner than that of lesser-quality roads.

The density to which a roadbase can be compacted is a function both of the moisture content of the aggregate and of the compacted effort exerted upon it. Moisture content is critical. If there is too little water, particles will not have the lubrication necessary to compact properly, regardless of compactive effort. With too much moisture, hydraulic forces develop that may actually force the particles apart under compaction. The limits are strict. A deviation of only 1 percent from optimum moisture may reduce density by over 2 lb/ft^3 and increase voids by as much as 8 percent.

Benefits

Compared with plain water, calcium chloride possesses a stronger moisture film due to its greater surface tension, reduced vapor pressure, and lower freezing point. These properties, in addition to the chemical's moisture-attraction capability and its ability to resist evaporation, provide seven benefits in a program of roadbase stabilization.

Greater Density

Research and field tests have shown that adding the proper amount of calcium chloride to the roadbase aggregate (usually 0.5 percent by weight) results in a greater density in the aggregate than would be achieved by the use of water alone. A calcium chloride solution has a stronger moisture film, which enhances the lubrication effect with less moisture. The aggregate components can then slide together easily as they are mechanically compacted.

Less Compactive Effort

There are two ways to look at this benefit: (a) less compactive effort to achieve specified densities, and (b) greater densities achieved with the same compactive effort.

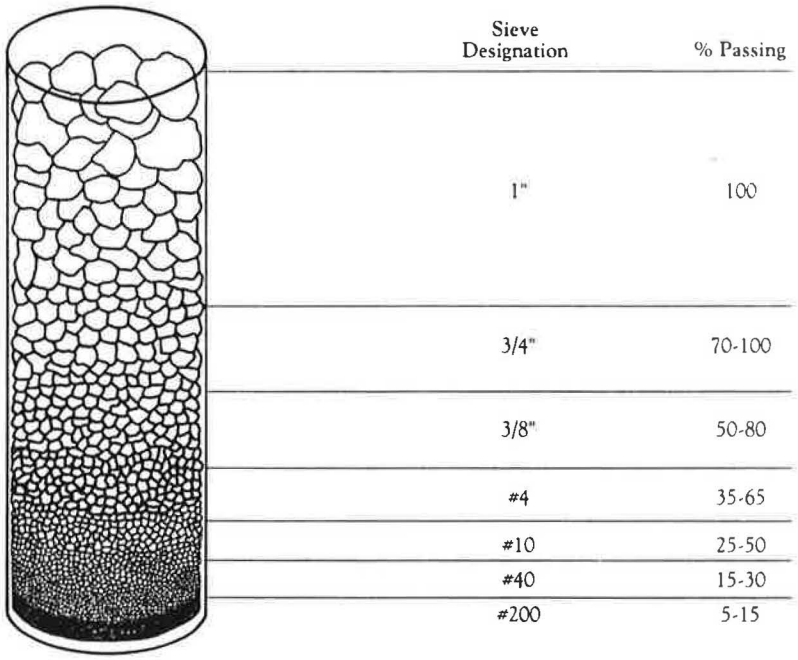

FIGURE 3 Average gradation of stabilized mix for typical base course.

In one experiment, a greater density was achieved at the end of four rollings when calcium chloride had been added to the aggregate than had been achieved with nine rollings when only plain water had been added.

Figure 4 shows that the use of calcium chloride can greatly reduce the compactive effort required to achieve specific densities.

Optimum Moisture Control

As previously noted, unless moisture is held within optimum limits, adequate densities cannot be achieved. An unstable roadbase would result, regardless of grading or the number of rolling passes made.

It is difficult to provide an aggregate mix with the proper amount of moisture. Once achieved, it is equally difficult to maintain optimum moisture content. Calcium chloride's hygroscopic qualities, along with its ability to lower the vapor pressure of water, work to inhibit evaporation and therefore help maintain optimum moisture (see Figure 5).

Surface Uniformity

Because of calcium chloride's moisture-retention capabilities, surface irregularities can be graded out and recompacted without loss of moisture content due to aeration and drying out. This can result in a smoother surface and provide a roadbase that is more uniformly dense throughout its depth.

Effective Stage Construction

When roads are being constructed in stages, treating the aggregate with calcium chloride can help the surface remain firm and stable. The need for frequent blading and aggregate replacement is thus reduced, as well as the costs involved with those types of maintenance.

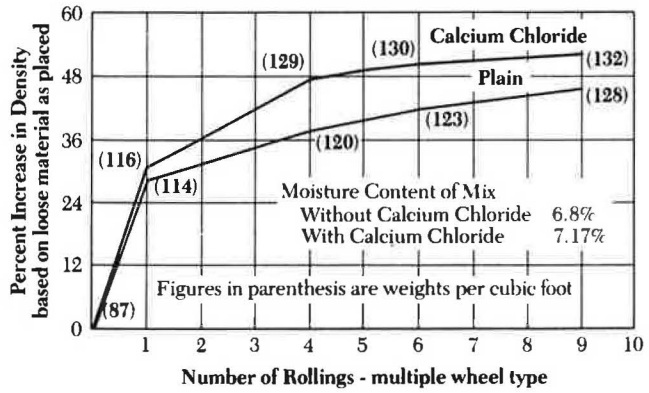

FIGURE 4 Compaction test results using calcium chloride versus plain water.

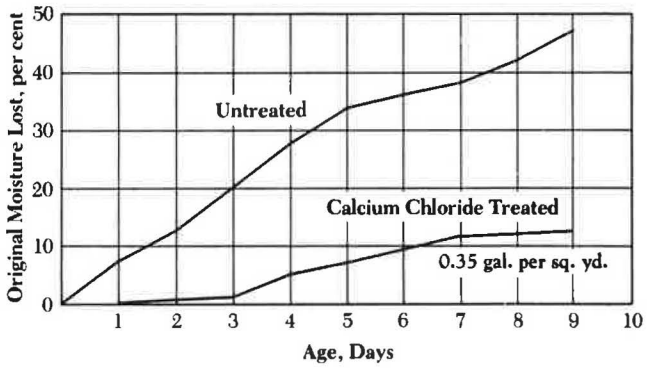

FIGURE 5 Moisture retention on road materials treated with calcium chloride versus untreated road materials.

Improved Bonding Between Base and Priming Materials

Because calcium chloride helps keep aggregate materials moist when they are being readied for paving, it helps improve the bond strength between the priming materials and the base course.

Frost Protection

The ability of calcium chloride to depress the freezing point of water helps a roadbase resist frost heave in winter. In tests using well-graded aggregate, only ½ of 1 percent calcium chloride by weight can in certain circumstances eliminate frost heave (see Figure 6).

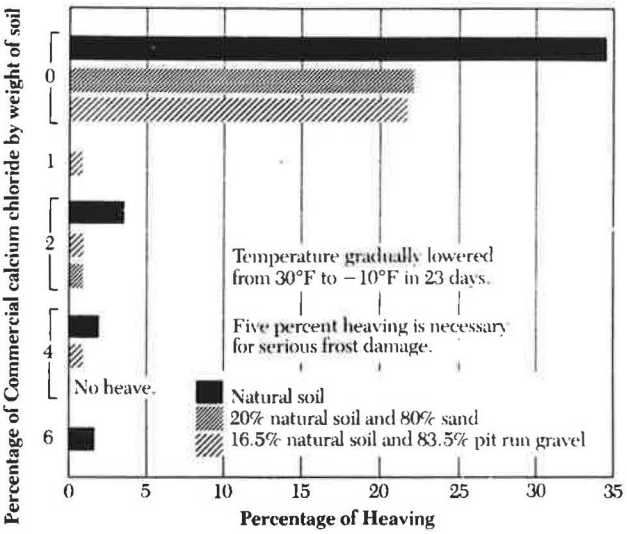

FIGURE 6 Effect of various amounts of calcium chloride on reduction of frost heaving in different soil mixtures.

Recommended Guidelines and Application Rates

Roadbase stabilization requires a total of 0.6 gal/yd² of liquid calcium chloride: 0.4 gal are used for stabilization, and 0.2 gal are used as a top dressing.

The following seven steps are needed to stabilize roadbases with calcium chloride.

Step 1. The road surface should be scarified to a depth that removes all potholes and other irregularities—usually a minimum of 6 in.

Step 2. If it is necessary to add aggregate to improve gradation, materials comparable to those already in place should be used. The material should be mixed with the existing aggregate at the work site. No more than 6 in. of loose aggregate at a time should be placed in a layer during the process of rebuilding the road surface.

Step 3. A 38 percent solution of liquid calcium chloride should be applied uniformly to the mixed or scarified material at the rate of 0.4 gal/yd². Alternatively, 2.27 lb/yd² of flake calcium chloride or 1.95 lb/yd² of pellet calcium chloride can be applied.

Step 4. The soil, new aggregate (if added), and calcium chloride, plus water (if necessary), should then be thoroughly mixed. This mixing can be easily accomplished with a motor grader. Mixing should begin as soon as possible after the calcium chloride is applied. The mix depth is the same as the scarification depth—up to a maximum of 6 in.

Step 5. The surface should then be bladed, shaped, and compacted to a straight-line slope of ½ to12 in.—a type "A" crown. On curves, the slope should remain the same across the entire width of the road (1½ to 12 in.). The transition between a straight line and curve should be gradual. As previously explained, this action allows water to drain off the road.

Step 6. The road should be top dressed by applying a 38 percent solution of liquid calcium chloride to the surface at the rate of 0.2 gal/yd². Alternately, 1.14 lb/yd² of flake calcium chloride or 0.97 lb/yd² of pellet calcium chloride can be applied.

Step 7. Once the road is stabilized, dust control is usually maintained throughout the summer with minimal attention. However, for best results, a second top dressing is recommended in late summer or early fall. A 38 percent solution of liquid calcium chloride should be applied to the road surface at the rate of 0.27 gal/yd². Alternately, 1.54 lb/yd² of flake calcium chloride or 1.32 lb/yd² of pellet calcium chloride can be applied.

If necessary, the surface should be rebladed according to Step 5 before applying the calcium chloride.

SUMMARY

When used for dust control, calcium chloride holds fines in place, which in turn holds the coarse aggregate in place. This reduces aggregate replacement costs. It also reduces the frequency of blading from 25 to 75 percent, which cuts labor and equipment costs. Fewer spot repairs are required, which means less fuel and less equipment maintenance are needed.

When added to a well-graded aggregate mix for roadbase stabilization, calcium chloride helps keep moisture at an optimum level, resulting in greater densities with less compactive effort, and less dust. It also causes a better bond between the roadbase and the surface course, providing effective stage construction and frost protection.

When these advantages are combined, the life of a paved road can be expected to double and, depending on the specifications, the cost can be reduced by approximately 30 percent per mile.

In short, whether calcium chloride is used for dust control or roadbase stabilization, it is an inexpensive chemical whose cost-saving benefits easily outweigh its cost.

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Soil Bioengineering—An Erosion Prevention Technique Applicable to Low-Volume Roads

LEONARD M. DARBY

Soil bioengineering is one of several new and innovative engineering practices being used on the FHWA Cumberland Gap tunnel project to prevent erosion and provide permanent stream bank protection. Four installations of soil bioengineering, using primarily sandbar and black willow cuttings, were constructed to provide stable and aesthetic creek bank protection and stream redirection for Little Yellow Creek in the Cumberland Gap National Historical Park. These four installations included a live boom (stream redirection), a brush mattress, live stakes, and joint planting. During the first 6 months, the dormant, primarily willow stems and trunks in all four installations sprouted new branches, leaves, and extensive roots that retained the soil and protected the banks in an effective and aesthetic manner. Over the next 2 years, the plant material weathered an officially recorded drought, insect attack, extreme heat and cold, and moderate flooding. Throughout these attacks, the plants showed remarkable resiliency and continued to expand vertically and horizontally both above and below the ground surface. After 3 years of growth, healthy plants up to 6 ft tall are maintaining aesthetic stream bank protection and their root system has stabilized the soil that makes up the stream bank slopes.

The Cumberland Gap tunnel project consists of twin, 4,600-ft-long, two-lane highway tunnels and portal buildings, eight roadway and two pedestrian bridges, 5 mi of four-lane divided highway, 3 mi of local two-lane and park roads, along with several miles of low-volume access roads and pedestrian walkways, including parking areas. The design and construction management is being done by the Eastern Federal Lands Highway Division (EFLHD) of FHWA headquartered in Sterling, Virginia, through their Cumberland Gap project office located in Middlesboro, Kentucky. The total project price tag is approaching $250 million and the work is broken down into about 25 separate contracts. The most current design and construction techniques and innovative engineering practices are being incorporated into the project by the EFLHD where the FHWA Demonstration Projects Division originated back in 1968. The project work is entirely in the Cumberland Gap National Historical Park, dictating maximum erosion control and pollution prevention compliance. The project includes a combination of complex high-volume road installations and basic low-volume road installations. All of the work has some, if not significant, application to low-volume roads, especially in the erosion prevention and stream bank protection area.

FHWA, Eastern Federal Lands Highway Division, P.O. Box 15, Harrogate, Tenn. 37752.

Low-volume roads will be considered to be those that generally have the following cost-effective design and construction characteristics

- Maximum use of readily available and easily obtainable material,
- Can be built using conventional construction equipment, and
- Maximum constructibility with simple designs.

Soil bioengineering is a new and innovative erosion prevention technique successfully used in the construction of the Cumberland Gap tunnel project that meets the cost-effective low-volume road design and construction criteria. Its specific use was stream bank protection and stabilization to prevent bank erosion. Conventional stream bank protection and stabilization has generally been to heavily reinforce the stream banks with moderate to heavy, 100- to 300-lb maximum size, compact riprap. This usually tedious operation requires many pieces of equipment to haul and place large quantities of quarry rock in unstable marshy areas adjacent to creeks. It is often an expensive operation because of the length that suitable riprap rock may have to be hauled and the cost of developing and maintaining a stable access road. Once completed, the riprap protection is usually as strong as it ever will be because the soil beneath the riprap only generates scrub brush with little to no reinforcing root system.

A decision to proceed with four soil bioengineering installations was made by EFLHD in concurrence with the National Park Service after a series of studies, meetings, and assurances by the soil bioengineering consultants.

The innovative technology and environmental aspects of soil bioengineering seemed best suited to fulfill the requirements for aesthetic, permanent stream bank protection along Yellow Creek in the Cumberland Gap National Historical Park near Middlesboro, Kentucky.

The four soil bioengineering units were installed on a newly constructed stream bank along Yellow Creek by prime contractor Melco-Greer, Inc., under CGNHP Project 25E7 during the winter of 1987. Two of these depended entirely on soil bioengineering for stream bank protection and stabilization. The third involved a combination of soil bioengineering and riprap to produce a combination system. The fourth was a special structure that redirected the stream and protected the redirected structure. It also used a combination riprap and soil bioengineering system. All four units were less than 100 ft long and used various lengths of willow branches

and trunks or a minimum Class 2 (100-lb-size) riprap. The 50-year design flow along Little Yellow Creek is 2,100 ft^3/sec.

Placed in layers in a mattress-like fashion along newly graded banks adjacent to Yellow Creek, a soil bioengineering unit comprised primarily of lengthy dormant willow branches and trunks termed a "brush mattress" was constructed as shown in Figure 1. Its purpose was to quickly provide vegetation capable of retaining critical soil along the bank on the west side of the creek. Once growth is established, the extensive network of willow roots should provide a soil retention cohesiveness that is both natural and living, growing stronger with each year's growth, potentially retaining more and more soil.

Advocates maintain that the brush mattress' continued growth ensures future soil retention qualities, in contrast to riprap, the other consistently used creek bank protection technique, which may be undermined as years pass.

The second unit incorporating soil bioengineering was a mound structure, termed a "live boom," and designated and constructed to redirect water flow away from vulnerable creek bank areas, as shown in Figure 2. The live boom is constructed from below the stream bed rising through the water and protruding several feet above the water, to a height capable of ensuring maximum water deflection to protect adjacent critical creek banks. This soil bioengineering unit was a time-intensive endeavor. Like most soil bioengineering units, it must be performed while the plants are dormant, usually after the first frost in the fall and before the sap rises in the spring.

A third unit of soil bioengineering constructed was termed a "live stake." A live stake is a dormant willow stake driven randomly on about 18- to 24-in. centers throughout the area designated to be live-staked. The live stake is about 2 ft long and ½ to 1½ in. in diameter. The cut or basal end of the

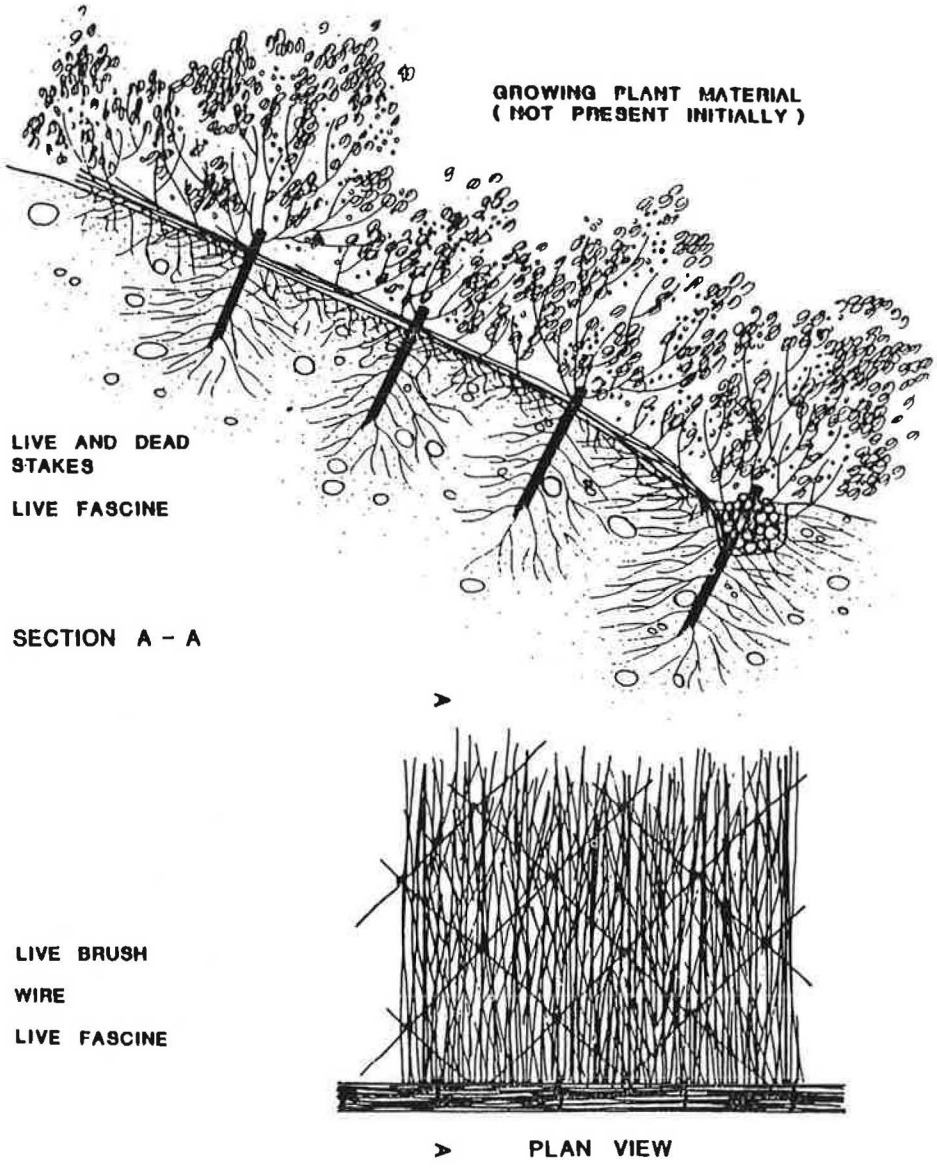

FIGURE 1 Brush mattress.

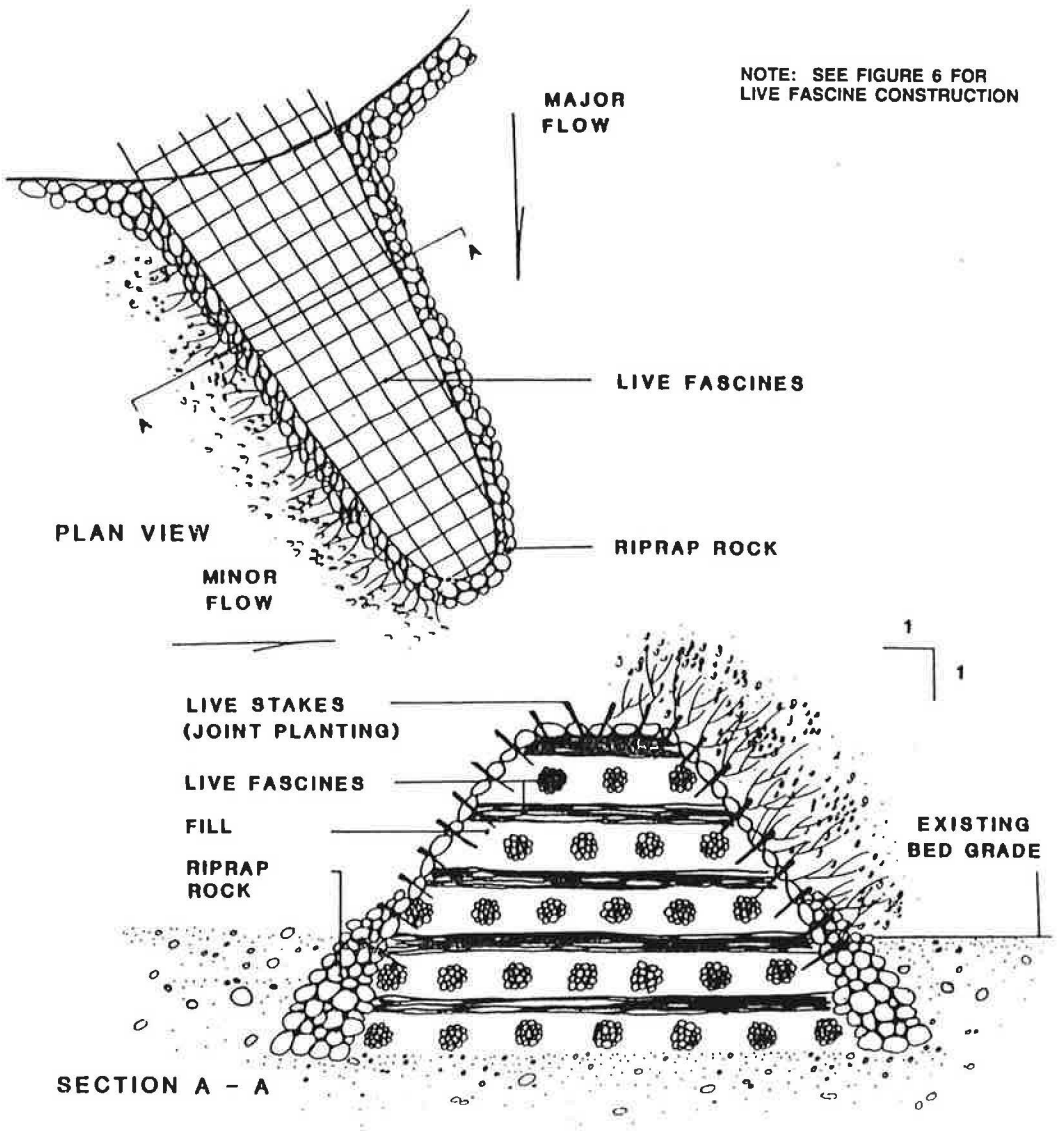

FIGURE 2 Live-boom construction.

stake, normally the rooted end, is driven into the ground, and if properly installed, will tie the bank soil together through an extensive network of below-ground-level roots and a prolific growth of above-ground-level leaves and branches as shown in Figure 3.

The fourth unit of soil bioengineering constructed is termed "joint planting." Joint planting is basically the placement of live stakes that are driven through a riprap layer to tie the underlying earth bank, surface riprap layer, and plants together in a living protective network as shown in Figure 4.

MATERIALS, EQUIPMENT, HANDLING, AND STORAGE

Materials used consisted of several rolls of hay baler twine; wire, similar to rebar tie wire used in concrete; hundreds of 2-ft × 2-in. × 4-in. diagonal wooden stout stakes (see Figure 5); hundreds of 2-ft live stakes consisting of 2-ft lengths of dormant willows ½ to 1½ in.; and 6- to 9-ft lengths of live, but dormant, primarily willow trunks and branches ½ to 1½ in. in diameter for making live fascine bundles and constructing brush mattresses. Ninety percent of the plants were sandbar or black willows. The remaining 10 percent were swamp poplar, common alder, and swamp dogwoods.

Equipment used consisted of the following: 2-ton, medium-duty, C60 Chevrolet flatbed truck; Caterpillar 235 track-mounted hydraulic excavator; three chain saws; five loping shears; three 3-lb impact-absorbing hammers; several sledgehammers; and 1 Homelite 3-in. water pump.

Two handling techniques, cutting and transportation, were used. Live but dormant plant material is cut and handled with care to avoid bark stripping and trunk wood splitting. Cuts are made 8 to 10 in. from the ground when cutting from the approved, natural growing, source sites. Cuts shall be made flat or at a blunt angle to ensure that the source sites will

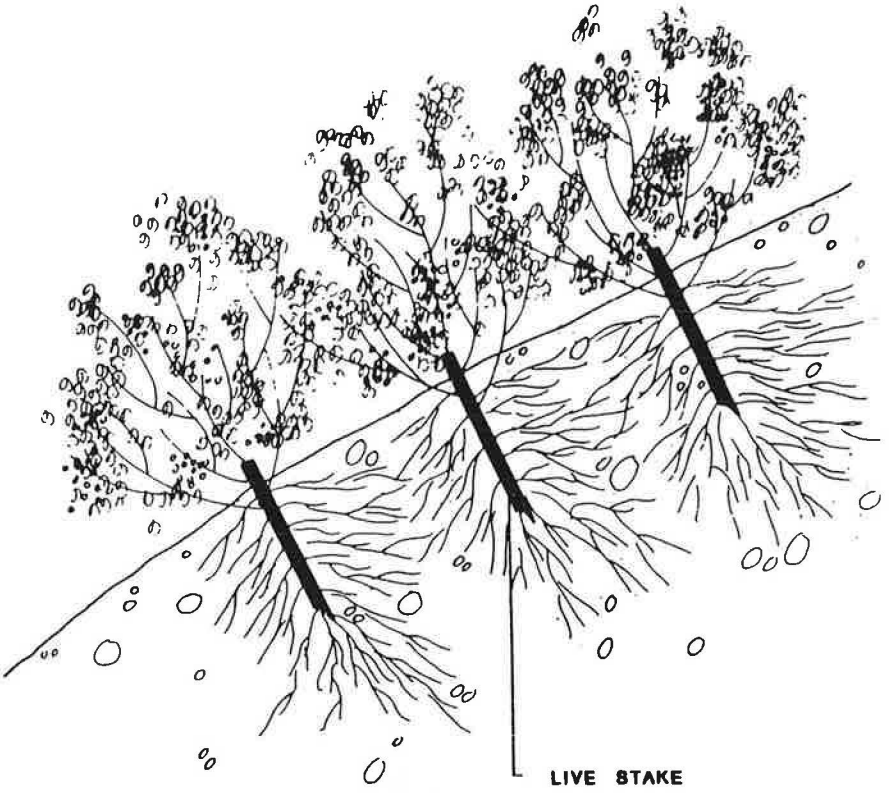

FIGURE 3 Live-stake construction.

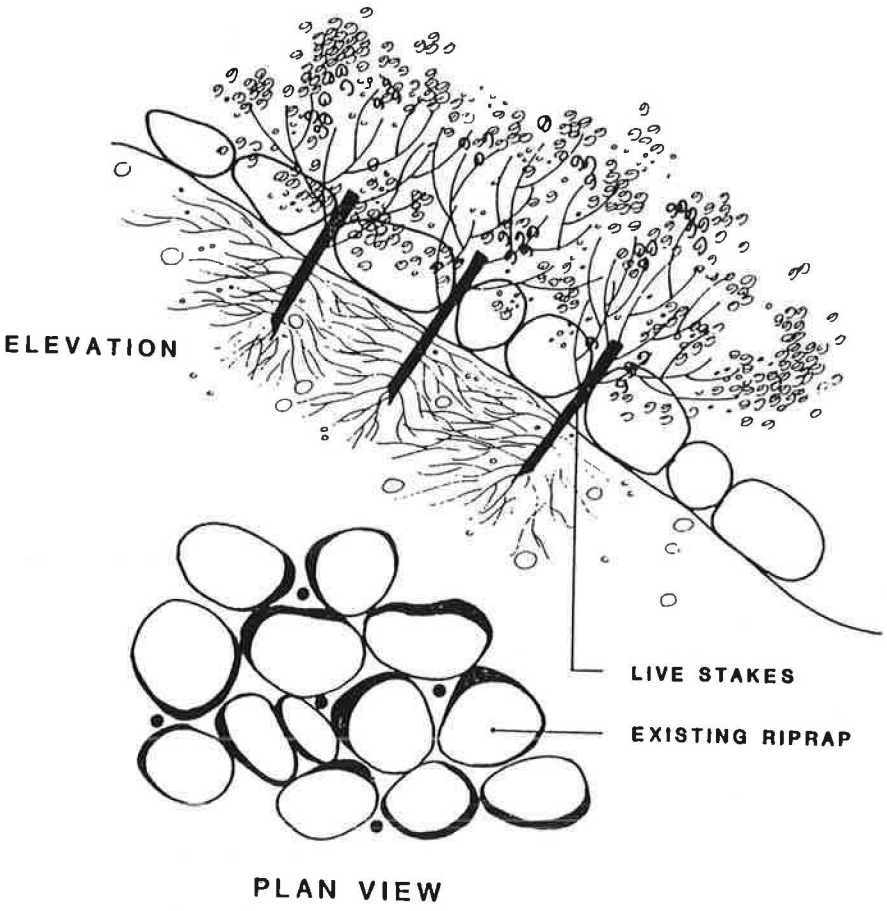

FIGURE 4 Joint planting.

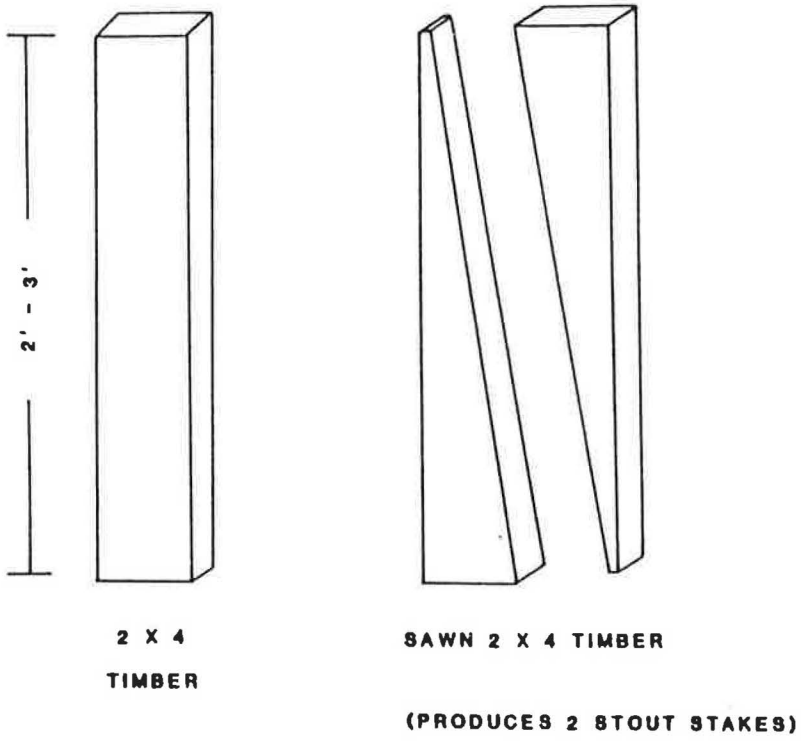

FIGURE 5 Stout stakes.

regenerate rapidly. During transportation, the live but dormant cut branch groups are placed on the transport vehicles in an orderly fashion to prevent damage and facilitate handling. The live but dormant cut plant materials shall be covered with a tarp or burlap material during transportation.

Plants not installed on the day of arrival at the job site shall be stored and protected. Outside storage locations shall be continually shaded and protected from the wind. Live but dormant cut-plant material shall be heeled-in in moist soils, or kept in water. Live but dormant cut materials shall be protected from drying at all times. When the temperature is 50°F or above, the live-cut branches shall not be stored but shall be planted on day of arrival. However, when the live-cut branches have been prepared in fabricated building lengths, such as for live-stake planting or similar uses, they shall be used that day. This prepared material may not be stored.

BRUSH MATTRESS CONSTRUCTION

During the stream channel construction, banks on the west side of Yellow Creek were realigned using 6-in. minus silty sand borrow material meeting AASHTO soil classification A–4 requirements, and were finished at a 2:1 slope. Except for areas reserved for brush mattress and live stakes, the slopes received a covering of polypropylene support (filter) fabric over which was applied a 2-ft layer of Class 2 (100-lb nominal size) limestone riprap near the outlet of a triple 10- × 10-ft reinforced concrete box culvert.

Before installation of the brush mattress, the slope receiving the brush mattress was raked and fertilized. Also, two trenches, approximately 8 ft apart and parallel to the stream, were dug to a depth of about 1 ft. Willow branch and trunk material called "live brush" were laid flat against the ground with the large cut (basal) end of the willow brush material pointing toward the creek with the smaller end pointing up the slope. The live brush material, spread evenly along the slope, was then wired to wood stakes driven on 2-ft centers to form a brush mattress. Wire was then interwoven in a zig-zag fashion and capable of holding the brush mattress closer to the ground. Sledge hammers were used for tamping the stout (nonliving) stakes deeper into the ground to ensure greater soil contact with the brush mattress and increasing the potential for quick willow growth.

Two long-life fascines, 6-in. diameter × 100-ft-long bundles, were dropped into the two previously dug trenches as shown in Figure 6. These bundles cover the cut ends of the willows along the bottom of the slope and along the middle of the slope, if two lengths of willows are used.

Afterward, this material was completely covered with topsoil, permitting a thin layer of exposure to facilitate the sun's ability to bring forth the willow growth. Small branch nodules from the cut willows lying against the soil become roots while branches facing the sun become stems. Good soil contact must exist to initiate and maintain growth. In fact, good soil contact must be maintained before completion of the brush mattress in accordance with the contract requirements. This is accomplished by uniformly tamping the topsoil, without damaging willows, to fill all voids and air pockets.

LIVE-BOOM CONSTRUCTION

The second phase of the soil bioengineering work encompassed the use of another living structure known as a "live

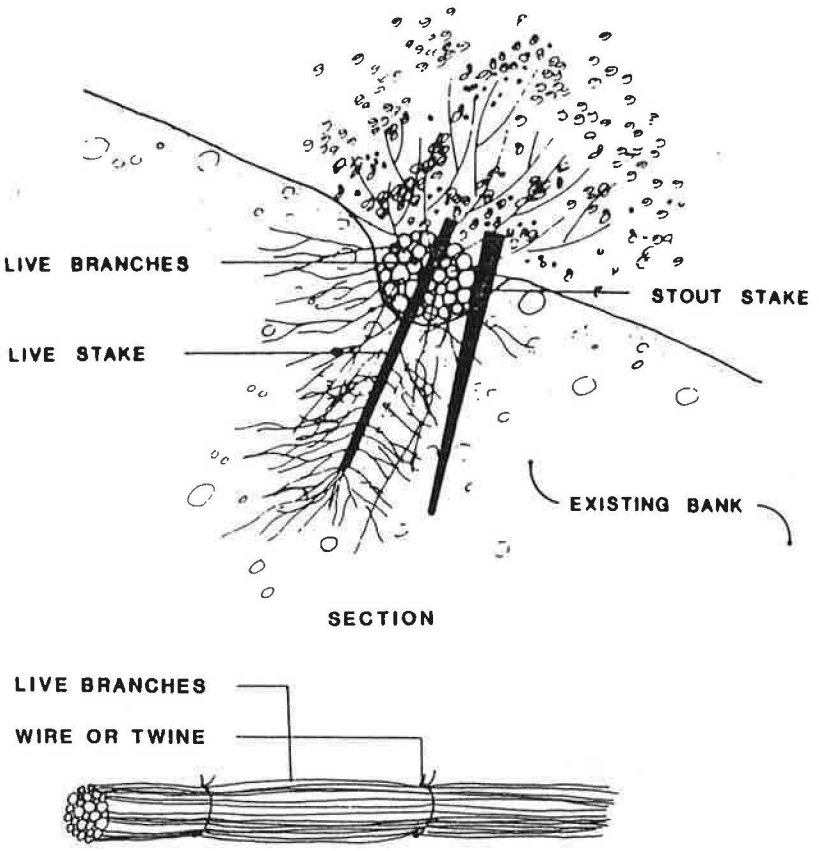

FIGURE 6 Live fascine bundle.

boom," shown in Figure 2, to effect stream redirection. The live boom begins as an area excavated within the stream bed and ends as a mound-like, triangular-shaped structure constructed of multiple layers of live but dormant willow material, termed "live fascines," similar to the ones used in the brush mattress. Each successive layer was covered with 6 in. of topsoil. However, that area below the creek was constructed using alternating layers of 6-in. maximum crusher run stone instead of soil, because the living material below the creek is not expected to grow, but merely deteriorate, providing additional plant food for living rooted material above.

The bullet-shaped live boom was constructed as an integral part of the surrounding creek bank in order to provide support to the live boom. The bullet-shaped structure was completed by driving live stakes in close proximity, on 1- to 2-ft centers, throughout the length of the live boom. Hand-placed riprap was used to armor the live boom to protect it from erosion until the willow population became established. Live stake and live fascine root growth will combine to create the live boom's soil erosion qualities. When heavy rains cause the creek to rise, the currents are deflected by the live boom while preventing erosion of critical creek bank areas. When it is not working to control the adverse effects of heavy stream flow and redirecting the stream, the live boom performs as an aesthetically pleasing ecological biome that becomes an integral part of its environment.

LIVE-STAKE CONSTRUCTION

An integral part of soil bioengineering is the use of cut primarily willow trunks and branches approximately ½ to 1½ in. in diameter and 2 ft long. These willow trunks are termed "live stakes." While still live but dormant, they are driven perpendicularly into the ground being live-staked. Live stakes are trimmed with an angled cut at the bottom or normally rooted end of the stake. The top of the stake is flat as a result of harvesting. Hence, the flat portion of the stake is tamped with an impact-absorbing hammer driving the stake deeply into the soil, exposing about 5 in. of the top portion of the stake. This unit was only used sparingly along the top portion of the creek bank to tie the brush mattress and joint planting units into the existing terrain.

If properly installed, when spring budding occurs and the sap is on the rise, the live stakes will grow into an extensive network of roots and branches. Because the live stakes are installed on about 18- to 24-in. centers throughout, growth should provide an aesthetically pleasing ground cover, as well as an interlocking network of roots.

JOINT PLANTING CONSTRUCTION

Joint planting is basically live staking, which is used in riprapped areas. This application is almost identical to live staking because

it uses willows of the same diameter as live staking (½ to 1½ in.) with the length being increased by approximately the thickness of riprap. Also an iron rod with a diameter slightly smaller than the willow stake can be used to drive a pilot hole to be filled with a joint plant.

COST

The single live boom contract lump sum bid price was $2,000. However, the contractor maintains that being unfamiliar with labor-intensive soil bioengineering techniques and installation procedures, he used an unexpectedly large number of man-hours, which resulted in a huge (factor of 10+) cost and time overrun in the construction of the single live boom. It took several weeks to construct rather than the several days the contractor had estimated.

Approximately 400 yd^2 of brush mattress was constructed in a single day's time after the harvest of the willows requiring less time and manpower than expected. The contractor reported that the brush mattress cost less than the $20.00/$yd^2$ unit bid price that was based on an estimated quantity of 600 yd^2. However, on subsequently bid CGNHP Project 25E9, the average of seven contractor unit bid prices was $25.19/$yd^2$.

The joint planting was constructed at 2-ft centers throughout the riprap area. The contractor received his unit bid price of $15.00 for each of the approximately 3,500 joint stakes planted. The contractor readily admitted that this endeavor cost much less than the $15.00 each, unit bid price, that was based on an estimated quantity of 2,500. Approximately 100 joint plants were installed each single shift day. Subsequent CUGA Project 25E9 bid prices from seven contractors averaged $7.85 each, based on an estimated quantity of 1,700.

Live (dormant) stakes driven into designated areas along the top of the creek bank (not between riprap) were paid for at the unit bid price of $10.00 each, whereas their furnishing and placement cost much less. Live stakes and joint stakes are nearly identical, the only difference being that the live stakes are directly installed into the ground, whereas the joint plantings are installed into the ground through riprap layers. Approximately 100 live stakes were installed each single-shift day. Subsequent CUGA Project 25E9 unit bid prices from seven contractors averaged $6.88 each for the estimated quantity of 6,500.

CONCLUSION

In the Cumberland Gap National Historical Park, four installations of soil bioengineering using primarily willow cuttings were utilized to provide stable and aesthetic creek bank protection and stream redirection. These four included a live boom (stream redirection), a brush mattress, live stakes, and joint planting. During the first 6 months, the dormant (primarily) willows in all four installations vigorously grew from a dormant state to a flourishing vegetative system beginning to retain the soil and to protect the banks in an effective and aesthetic manner. Root systems were found to exceed the surface trunk and branch system. Over the next 2 years, the plants weathered an officially recorded drought, insect attack, extreme heat and cold, and recent high water. Throughout these attacks, the plant materials showed remarkable resiliency and continued to expand vertically and horizontally both above and below the ground surface. After 3 years of growth, healthy plant materials up to 6 ft tall are maintaining aesthetic stream bank protection while the root system has stabilized the soil that makes up the stream bank slopes.

Guidelines for Handling Acid-Producing Materials on Low-Volume Roads

THOMAS W. FENNESSEY

In certain geologic formations, the use of conventional excavation and embankment construction techniques may result in acid runoff (pH $\leq$ 4.5). In turn, this may contaminate nearby streams and severely impact the fauna and flora that live in those streams. Acid runoff is most likely to be produced by the weathering of rock materials containing sulfur mineralization in excess of 0.5 percent that do not contain sufficient alkaline mineralization to neutralize the resulting acid runoff. Often, low-volume roads are constructed in environments where live streams and wildlife exist close to the construction right-of-way. Accordingly, environmental impact of the construction of low-volume roads on the adjacent lands is of great concern. Guidelines are presented that have been developed and implemented on existing low-volume roads to minimize the impact that the handling of acid-producing materials has on the adjacent environment.

As encountered in conventional roadway excavation and used herein, acid-producing materials are defined as those materials which, when exposed to the weathering process, produce acid runoff having pH values of 4.5 or less. Typically, it is the fresh exposure of unweathered rock that creates the potential for acid runoff. Soil, as a product of the decomposition and leaching caused by the weathering process, does not normally pose a threat to produce acid runoff unless the soil still contains a significant amount of undecomposed rock particles.

Rock materials most susceptible to producing acid runoff upon weathering are normally those that contain 0.5 percent or more of sulfur mineralization and that do not contain sufficient alkaline mineralization to neutralize the resulting acid runoff. The sulfur mineralization is most commonly found in the form of pyritic sulfur. The sulfur mineralization can be found in both venous and disseminated form.

Potentially acid-producing geologic formations are found in all three classes of rock; sedimentary, metamorphic, and igneous. However, the most common acid-producing materials occur as sedimentary and metasedimentary deposits such as carbonaceous shale and argillaceous deposits that have little or no neutralization potential. Sulfur mineralization can also be found in coarser textured sandstone and metasandstone. The presence of sulfur mineralization within these materials is generally attributable to the anoxic environment under which the sediments were deposited (*1*).

The darker color commonly associated with carbonaceous materials may serve as an indicator of potentially acid-producing mineralization. However, it is not a sure indicator. All materials in a geologic formation deposited under an environment conducive to the formation of sulfur mineralization should be suspect.

FHWA, Eastern Federal Lands Highway Division, 21400 Ridgetop Circle, Sterling, Va. 22170.

Currently, laboratory testing methods have been established to determine the acid-producing potential of a suspect material (*2*). The methods evaluate the acid-producing potential and the neutralization potential of a material in terms of equivalent tons of calcium carbonate per 1,000 tons of material. The result is commonly expressed as the net neutralization potential. A material having a net neutralization potential of 5 tons or less (a net acid-producing potential of 5 tons or more) is considered to be capable of producing environmentally harmful acid runoff. This material, therefore, requires special handling and treatment during excavation and embankment construction to minimize the impact on the adjacent environment.

In reality, the process of handling acid-producing materials begins well in advance of, and extends well beyond, construction itself. It is best expressed as a four-phase process—

- Preliminary design,
- Final design,
- Construction, and
- Postconstruction.

The process discussed involves expenditure of significant effort with an increased cost to the project. However, the purpose of this process is to avoid the additional cost to the environment and public opinion or perception associated with damage to environmentally sensitive areas adjacent to low-volume roads.

PRELIMINARY DESIGN

The first phase of this process focuses on determining whether or not acid-producing materials underlie the proposed roadway corridor. This phase begins with examination of available geologic references regarding the type and character of the rock formations that underlie the project site. Chemical analysis of the rock materials presented in geologic references may provide a quick assessment of the acid-producing potential of the rock formations.

Examination and sampling of exposed outcrops should also be done along the proposed corridor. These samples, along with rock core samples from a preliminary subsurface investigation, should be tested to determine the net neutralization potential of suspect materials. Geologic stratigraphy and structure should be noted to determine whether potentially acid-producing materials may only be encountered at certain elevations or limited areas along the project. At this stage in the project, it is important to be flexible to consider an align-

ment shift (vertical or horizontal) or an alternate alignment corridor to reduce, or avoid, exposure of acid-producing materials.

FINAL DESIGN

If acid-producing materials are indicated to underlie the project corridor during the preliminary design phase, additional steps will be required during the final design phase to address and prepare for the handling of the materials. The first step to be undertaken is the implementation of a water quality monitoring program. The accumulation of a set of baseline data, with regard to water quality in adjacent streams and ponds, is a time-consuming, yet essential item. Once established, the water quality monitoring program will be a continuing activity through the construction and postconstruction phases in the process of handling acid-producing materials.

It is also recommended that a more detailed subsurface exploration be performed to more accurately define the location and amount of acid-producing materials that will require special handling techniques. It is readily apparent that the subsurface exploration program should examine the materials to be encountered over the full depth of excavation at proposed cut locations. It may not be so readily apparent that the subsurface exploration program should also examine the materials to be encountered at proposed fill locations where excavations will be made for fill benching. The use of conventional rock core drilling in subsurface exploration can be supplemented with the use of air drills. A composite sample of the cuttings from the air drill excavation can be obtained. These samples, as well as rock core samples from additional rock core drilling, should be chemically tested to determine the net neutralization potential of the materials.

Another tool to be used in the detection of rock containing sulfur mineralization is induced-polarization (IP) resistivity surveys (*3*). The basis of this technique uses the response characteristics exhibited when an electric current passes through a subsurface zone of mineralization. IP resistivity surveys can be used to provide a more continuous subsurface exploration method to examine the entire alignment as a supplement to a rock sampling and testing program. However, the IP resistivity survey technique must be calibrated with knowledge of the geology and chemical testing of rock samples because IP resistivity surveys cannot distinguish between the presence of sulfur mineralization and the presence of other good conductors such as graphite. In addition, the induced polarization survey acts only as a method to detect potential acid-producing mineralization and cannot assess the neutralization effects of alkaline mineralization in the same rock formation.

IP resistivity surveys can be run as both reconnaissance and more detailed subsurface exploration tools. Thus, the line interval, electrode interval, and electrode configuration must be chosen with a specific purpose in mind (*3*). For example, in a reconnaissance survey, an electrode interval of 200 ft with a single line interval may be desired, whereas for a more detailed survey, an electrode interval of 25 ft with multiple line intervals may give better resolution and detection of smaller mineralization deposits. The depth and width of the proposed cuts should be considered in setting the configuration of the IP resistivity survey.

The results of induced polarization resistivity surveys are commonly expressed in terms of percent frequency effect (PFE), metal factor (MF), and resistivity (R). Establishment of threshold values of PFE, MF, and R that correlate with the presence of acid-producing materials and the resulting net neutralization potential should be established for use on each project. Suggested threshold parameters (*1*) include—

- PFE values greater than 10 percent generally indicate high concentrations of polarizable mineralization,
- MF values greater than 1 generally indicate easily weathered polarizable mineralization, and
- R values less than 10 000 ohm-m generally indicate easily weathered materials.

These parameters should be compared with the results of chemical analyses of available rock samples. However, good correlation may not occur for the reasons indicated previously.

As the more detailed information on the location and extent of acid-producing materials along the alignment corridor becomes available, it is again time to evaluate whether it is appropriate to use an alignment shift or grade change to avoid or minimize excavation of acid-producing materials. It should also be pointed out that significant alignment changes at this point will likely warrant additional subsurface exploration.

Once the final alignment is determined, the quantity of acid-producing materials requiring special handling must be determined from project cross sections. For ease of construction, it may be determined to be more cost-effective to consider all materials from an entire cut area, including soil overburden, to be acid-producing rather than require selective excavation and handling by the contractor. Prudent design also includes providing a percentage of excess volume in the quantity of acid-producing materials as a factor of safety to accommodate additional small quantities of such materials not identified during the subsurface exploration process.

Once the quantity of acid-producing materials requiring special handling is determined, the method of handling these materials must be designed. Any effective handling method must achieve one or more of the following (*1*):

- Control oxygen,
- Control water,
- Promote alkalinity,
- Control acidophilic bacteria, and
- Remove sulfides.

Currently, encapsulation of the acid-producing materials is the favored method. Encapsulation seeks to eliminate groundwater flow through the acid-producing embankment materials by providing drainage provision beneath the fill. Encapsulation seeks to provide excess alkalinity by the inclusion of neutralizing materials within the acid-producing materials. Finally, encapsulation seeks to cap the acid-producing materials to cut off the infiltration of air (i.e., oxygen) and water into the fill, preventing formation of the proper atmosphere for the production of acid runoff.

Based on the design method employed and the amount of acid-producing materials to be encapsulated, the amount of materials to be used in the encapsulation process must be quantified. Sources for these encapsulation materials must be

located and identified. Provisions must then be made to locate and design the individual sites for the encapsulation of the acid-producing materials. The encapsulation sites can be located either on-site or off-site in embankment or side-hill fill locations. The sites should be selected to minimize haul distances of the acid-producing materials and the encapsulation materials.

Drainage is another area of special concern in the final design phase. Surface water should be directed away from cut slopes in acid-producing materials and away from encapsulation sites. Paved waterways should be used in the ditches above encapsulation sites to minimize infiltration. Curbing can also be used at the edge of pavement above encapsulation sites to reduce sheet flow down the face of the slope and further minimize infiltration. Ditches below cuts in acid-producing materials should be paved or limestone lined to act to neutralize minor acid runoff. Encapsulation sites should be selected to avoid high groundwater or stream flow locations.

In order to minimize the surface area of acid-producing materials exposed in cut sections, it is recommended that cut slopes in these materials be made as near to vertical as possible. On the other hand, the slopes of acid-producing fill materials and the encapsulating materials that cover them should be designed with a conservative factor of safety against slope stability failure. Where cut or fill slopes 1.5H to 1V or flatter are used with acid-producing materials, the slopes should be capped with soil, seeded, and mulched.

An encapsulation design employed for the past several years by the Federal Lands Highway Division of the FHWA on several low-volume road projects on U.S. Forest Service land is shown in Figure 1. The design calls for the placement of a 12-in.-thick drainage blanket of crushed limestone (AASHTO M43 No. 57 stone) bound top and bottom with a layer of filter fabric against the benches at the back and base of the fill slope. Six-inch underdrain pipes are incorporated into this drainage blanket at the back of each fill slope bench. Acid-producing materials are then placed in 2-ft-thick compacted lifts. Each lift is treated with 500 lb of agricultural lime per 1,000 ft^2. The encapsulation site is then covered with a 6-ft thickness of compacted soil (AASHTO Classification A–4).

On a recent project where limited quantities of on-site cover materials were available, this design was modified to reduce the thickness of the limestone drainage blanket and the soil cover by 50 percent to 6 in. and 3 ft, respectively. Evaluation of field tests simulating this modification indicates acceptable results (*1*). Monitoring of the actual installation is ongoing.

CONSTRUCTION

The most important element in the construction phase of this process is that the contractor and construction project engineer both be familiar with the known locations of, and procedures for, handling acid-producing materials. The contractor must be familiar with special handling provisions to efficiently schedule the work to promptly transport and place acid-producing materials in a designated encapsulation site as encountered. Unnecessary handling and storage of acid-producing materials should be minimized.

In addition, as materials are excavated, they should be examined to verify or detect the presence of sulfur mineral-

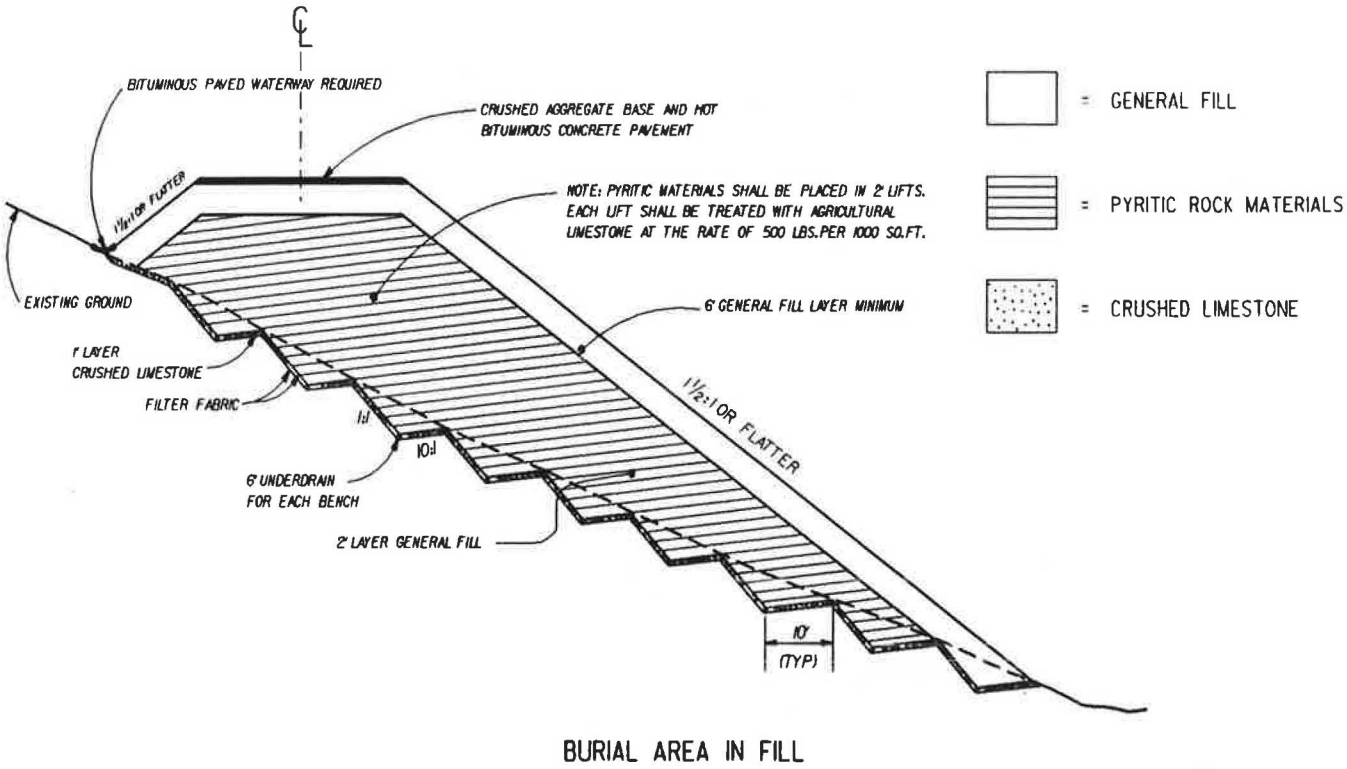

FIGURE 1 Method for treatment of acid-producing material.

ization during construction. In the field this should be done at least visually by a trained inspector. Any questionable material can be temporarily covered with plastic film or treated with lime until results of chemical testing are available to determine the appropriate final handling of the material. As a supplement to the visual inspection, composite samples of rock materials from air drills can be routinely obtained and chemically tested in advance of the excavation to aid in detecting any changes in the anticipated handling procedures. During construction, it is also critical to monitor and compare the actual versus estimated volumes of acid-producing materials coming from the cut areas. The values must be compared to determine if sufficient encapsulation site volume is available for the remainder of the project.

An ongoing program of water quality monitoring should already be in place at the time of construction. The frequency and location of such monitoring should be increased as construction is underway in an effort to quickly detect and correct any problems as they occur. The monitoring program should be structured in such a manner that water quality sampling frequency is tied to local precipitation events and not to a strict sampling schedule.

POSTCONSTRUCTION

The encapsulation method discussed previously has performed satisfactorily for the past several years and is considered to be a proven method. However, studies have shown that, even after 10 years of exposure, acid-producing materials are still capable of producing significant acid runoff (*1*). Thus, while the handling of the acid-producing materials are still capable of producing significant acid runoff (*1*). Thus, while the handling of the acid-producing materials ends with construction, the water quality monitoring program may be continued for a period of time to assess the postconstruction performance of the encapsulation and special handling design features. This is particularly encouraged where modifications or new methods of handling acid-producing materials are employed. The postconstruction water quality monitoring program can also include monitoring of discharge from drainage blankets beneath encapsulation areas and drainage in ditchlines below exposed cuts in acid-producing materials. As noted previously, such a monitoring program should be structured in such a manner that water quality sampling frequency is tied to local precipitation events and not to a strict sampling schedule.

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Geotechnical Engineering

Predicting Subgrade Moisture Under Aggregate Surfacing

ROBERT F. HINSHAW AND JIMMIE L. NORTHRUP

Methods and results of monitoring subgrade moisture under seven aggregate-surfaced roads in northern Idaho are presented. Data were collected over a 5-year period from 1978 to 1982. The data were collected to predict subgrade moisture for consideration of seasonal variations in subgrade strength in aggregate thickness design. One hundred forty-six specially calibrated electrical resistance moisture-temperature cells and 100 nuclear depth probe tubes were installed in the roads' subgrades. About 9,100 electrical cell observations were analyzed. Electrical cell measurements were within about 1 to 2 percent saturation of gravimetric (oven-dry) tests. Comparison between nuclear depth-probe measurements and gravimetric-volumetric tests was poor, and the method was abandoned about midway through the project. Saturation decreased from a maximum of about 95 percent in early spring to a minimum of about 65 percent in early fall, and then increased until freeze-up. Two regression equations for percent saturation that are based on electrical cell readings are given: one as a cosine function of time ($R^2 = 0.76$) and the other as a function of antecedent precipitation index ($R^2 = 0.78$).

The methods and results of field monitoring studies by the USDA Forest Service on subgrade moisture under typical aggregate-surfaced logging roads in northern Idaho are described. The general purpose of the studies was to find a method of predicting subgrade moisture to allow consideration of seasonal variations in subgrade strength for aggregate thickness design. In order to determine the validity of data, the studies included an assessment of the instrumentation accuracy. Once subgrade moisture is predicted, soil strength tests such as the California bearing ratio (CBR) can be run in the laboratory to simulate field conditions.

Many Forest Service logging roads are used only during a limited haul season. Typically, in the northern part of the country and in high mountainous areas, logging roads are closed during the winter and during the spring breakup period. The Forest Service Aggregate Surfacing Design Guide (*1*), which was adopted in 1990, uses varying subgrade soil strengths for increments of time within the design season. A similar procedure was developed in Region 1 of the Forest Service in 1980.

Data were collected from 1978 through 1982 in two study areas; Horse Creek in the Nezperce Forest and the St. Maries area of the Idaho Panhandle National Forests. The Horse Creek and the St. Maries studies differed in the intensity of instrumentation. The Horse Creek study was heavily instrumented over a 1.2-mi road segment. The St. Maries study was less heavily instrumented, but represented a wider variety of soil and climatic conditions on six roads totaling 31 mi.

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STUDY AREAS

St. Maries

The St. Maries project area is approximately 65 air miles southeast of Spokane, Washington, as shown in Figure 1. The roads are identified as Staples, Christmas, Merry, Gold, Emerald, and Elk. The majority of the test roads were constructed and surfaced with aggregate in the 1970s.

Topographic and climatic characteristics at the St. Maries test area are as follows:

- Elevation—2,800 to 5,000 ft above sea level;
- Topography—0 to 70 percent side slopes in mountainous terrain;
- Temperature—Average annual temperature is 48°F, average mean daily temperature is 68°F in July, and average mean daily temperature is 28°F in January;
- Precipitation—Average annual precipitation is 41 in. with 50 percent occurring as snow; and
- Vegetation—Moderately heavy fir and pine forest.

The roads are single-lane, aggregate-surfaced, logging roads with varying top widths. The aggregate surfacing was good-quality, dense-graded, crushed rock with depths of 4 to 12 in. Road grades ranged from 1 to 9.5 percent. The roads represent all four aspects. Major traffic on the roads is related to timber haul.

Subgrade soils are residual nonplastic silts and silty sands derived from the weathering of the underlying mica schists and quartzites. Mean index properties are presented in Table 1. Figure 2 shows an example of the results of laboratory tests run to determine the relationship between CBR and percent saturation to relate field data to seasonal changes in subgrade strength. A groundwater table does not exist in the soil layer overlying the bedrock. Groundwater is limited to seeps and springs at the surface and some fractured areas in the bedrock.

Horse Creek

The Horse Creek study was a satellite project identified as "Study Plan 6" within the Forest Service's Horse Creek Administrative Research Project. The overall research project is a long-term monitoring program begun in 1965 to deter-

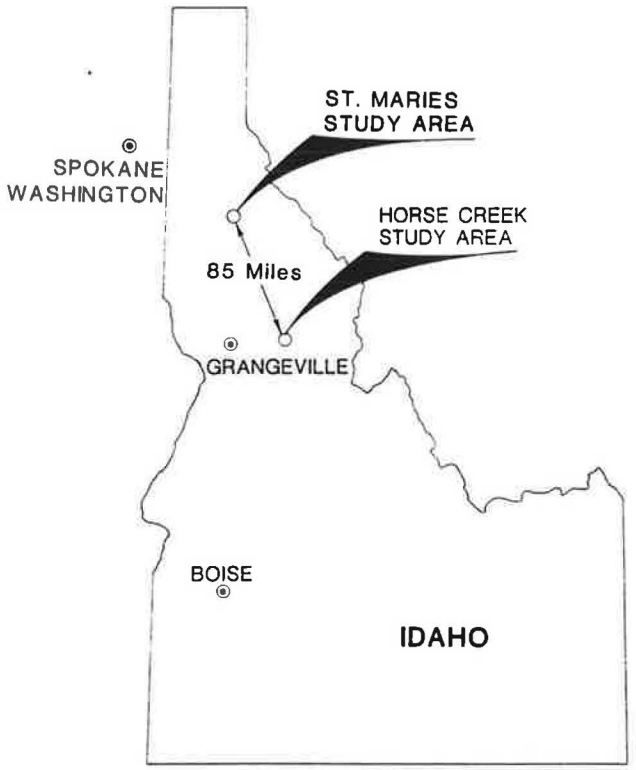

FIGURE 1 Project location.

mine the effects of logging and road building on water quality. The need for aggregate surfacing research was recognized in the late 1970s. It was decided to use similar techniques as developed at St. Maries and expand that effort because road construction and timber-hauling activities were carefully controlled, related weather data was being collected, and research personnel were available.

The test road is about 35 air miles east of Grangeville, Idaho, as shown in Figure 1.

Topographic and climatic characteristics at the Horse Creek test area are as follows:

- Elevation—5,500 ft above sea level;
- Topography—10 to 45 percent sideslopes near top of a ridge in mountainous terrain;
- Temperature—Average annual temperature is 37°F, average mean daily temperature is 60°F in July, average mean daily temperature is 23°F in January;
- Precipitation—Average annual precipitation is 49 in. with 72 percent occurring as snow; and
- Vegetation—Moderately heavy fir and pine forest.

The road is a single-lane, aggregate-surfaced logging road with a 12-ft top width. The 1.2-mi road was surfaced in three segments with 4, 8, and 12 in. of good-quality, dense-graded, crushed rock. Road grades range from −6.4 to +5.6 percent. Major traffic on the test road is related to timber haul.

Subgrade soil is nonplastic silty sand derived from metamorphic rocks of the Idaho Batholith border zone. It is locally called "decomposed granite." Soil is above the groundwater table. Mean index properties are presented in Table 1.

STUDY PLAN

St. Maries

At the St. Maries project area, 81 electrical resistance moisture-temperature cells (Soiltest Model MC-300B) were installed with the top of the cell approximately 4 in. below the top of the subgrade. All cells were installed in roads with existing aggregate surfacing. The cells were placed on the inside and outside wheel tracks. Moisture contents and soil temperatures

TABLE 1 MEAN SUBGRADE SOIL PROPERTIES (STANDARD DEVIATIONS IN PARENTHESES)

ROAD	Horse Crk	Staples	Xmas	Merry	Gold	Emerald	Elk
No. Samples	24	12	8	12	8	30	12
% - No. 4	89 (6)	92 (12)	85 (11)	88 (10)	88 (12)	91 (8)	93 (3)
% - No. 200	32 (6)	56 (15)	46 (12)	50 (8)	49 (9)	51 (18)	46 (6)
PI	NP	NP	NP	NP	NP	NP	NP
Unified Clas.	SM	ML	SM	SM-ML	SM	ML	SM
S.G	2.78 (.09)	2.69 (.03)	2.64 (.03)	2.63 (.05)	2.63 (.05)	2.71 (.08)	2.67 (.07)
AASHTO T-99 MAX Dens, PCF	113 (7)	112 (4)	110 (8)	102 (10)	104 (9)	107 (9)	105 (7)
Opt. Moist., %, AASHTO T99	16 (3)	15 (2)	15 (3)	18 (5)	17 (4)	16 (5)	16 (3)
In-Place Density, PCF	118 (8)	110 (7)	106 (12)	103 (15)	104 (12)	113 (12)	106 (7)
% T99 Max.	106 (6)	98 (4)	96 (8)	101 (7)	101 (4)	106 (11)	101 (4)

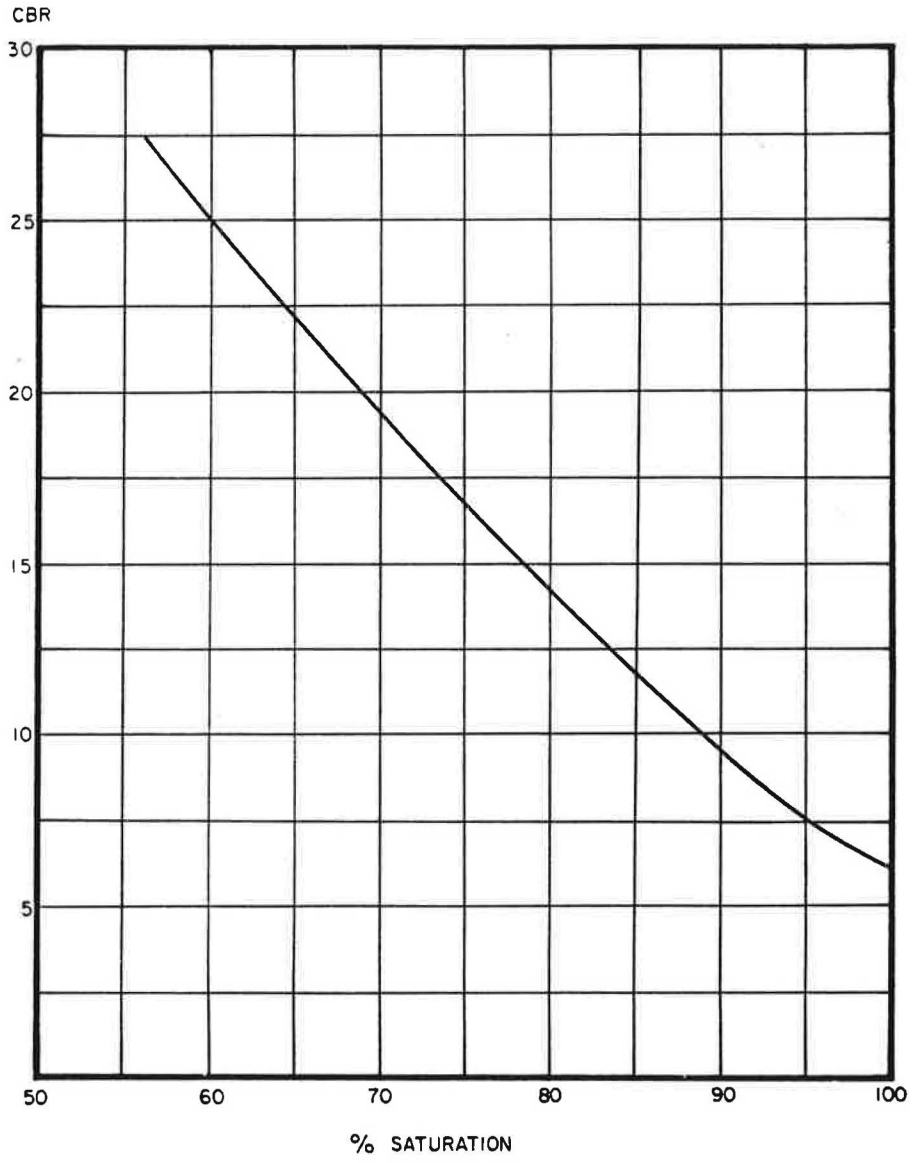

FIGURE 2 Example of a range of subgrade strength (CBR) values from a soil tested at varying moisture conditions.

were measured weekly during the April to November field season from 1979 through 1981 and less frequently in 1982. Approximately 3,620 observations were made.

Eighty-two nuclear depth probe tubes were installed in 1978, 1979, and 1980. The tubes were installed on the inside and outside wheel tracks within a few feet of the electrical cells. The subgrade was uncovered and readings taken with a Campbell Nuclear Pacific Model 501/504 nuclear gauge at 1.0, 1.5, 2.0, and 2.5 ft below the top of the subgrade. The readings were taken two to four times during field seasons.

Measurements of in-place density at the top of the subgrade were attempted in 1978, by digging out the overlying aggregate and using a direct-transmission nuclear gauge. However, the data were erratic, apparently because of the boundary effect of the aggregate walls of the hole. Eighty-one in-place densities were then taken volumetrically using a ring densometer (similar to a Washington densometer).

Horse Creek

At the Horse Creek area, 65 electrical resistance moisture-temperature cells were installed in the subgrade before the aggregate surfacing was placed. The cells were placed in groups of three or four in the vicinity of each of the 18 nuclear tubes discussed below and also in groups of two at 10 intermediate stations. Moisture contents and soil temperatures were measured weekly from 1978 through 1980, and less frequently during 1981 and 1982. Approximately 5,490 observations were made.

Eighteen nuclear depth probe tubes were installed in the subgrade soil at the end of construction in 1978. The locations were chosen to represent various conditions of road geometry. The subgrade was uncovered and readings of moisture and density taken at 0.5, 1.0, 1.5, 2.0, and 2.5 ft below the top of the subgrade at monthly intervals during the field season.

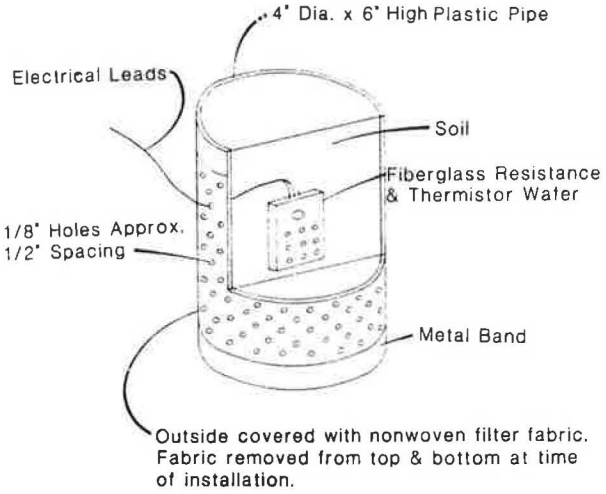

FIGURE 3 Electrical cell details.

Samples of the top 4 in. of subgrade were taken approximately monthly during the first 2 years of monitoring from the vicinity of the nuclear tubes for gravimetric (oven-dry) moisture contents.

Measurements of in-place density at the top of the subgrade were attempted in 1979 by digging out the overlying surfacing and using a direct-transmission nuclear gauge. The data were erratic and similar to the St. Maries experience. In-place densities were then taken volumetrically with a Washington densometer.

INSTRUMENTATION

Electrical Cells

The electrical cells are fiberglass wafer resistance devices calibrated to measure moisture content. The wafer also contains a thermister for measuring soil temperature. The devices had to be carefully calibrated in the laboratory in soil of the same type and density as exists in the field. Schematic diagrams of typical cell configuration and field installation are shown in Figures 3 and 4, respectively.

The calibration procedure for electrical moisture cells was as follows.

- The cells were saturated in water;
- While air drying, the cells were weighed periodically to determine moisture content and the corresponding resistance reading taken; and
- The saturation-drying cycle was repeated at least three times or until the calibration plot of resistance versus moisture content repeated itself.

To determine accuracy, a comparison was made of available gravimetric and electrical cell moisture data by paired *t*-tests. At Horse Creek, 250 gravimetric samples were taken at the top of subgrade in conjunction with electrical cell readings. Sixty-five gravimetric measurements of moisture taken at St. Maries during the initial stages of the project for in-place density determination were compared with the average percent saturation at the same site, on the same date (±3 days), in subsequent years. The results of the paired *t*-tests are as follows:

	Number of Pairs	*Mean Difference, Electrical Minus Gravimetric Measurements, Percent Saturation*	*Standard Error of Difference*	*Probability Level*
Horse Creek	250	−0.6	1.0	0.58
St. Maries	65	−0.01	2.0	0.996

The probability level is the probability of no difference between gravimetric and electrical readings. In general, electrical cell readings are within about 1 to 2 percent saturation compared to gravimetric measurements.

Nuclear Depth Probes

The configuration of the nuclear depth probes is shown in Figure 5. The aluminum tube holes were drilled with a truck-mounted rotary drill using a 2-in.-diameter saw-tooth bit on the end of a standard penetration test split spoon. The tube was a tight fit and normally required gentle pushing into place with the hydraulic action of the drill chuck.

At Horse Creek, a comparison of nuclear and gravimetric/volumetric measurements was made by digging out two of the tubes immediately after taking nuclear readings. Gravimetric moisture contents and volumetric in-place densities using a Washington densometer were determined from 0.5 to 2.5 ft below subgrade at each 0.5-ft interval corresponding to the nuclear measurements. A paired *t*-test was made of the difference between the nuclear and the gravimetric/volumetric measurements, with the following results.

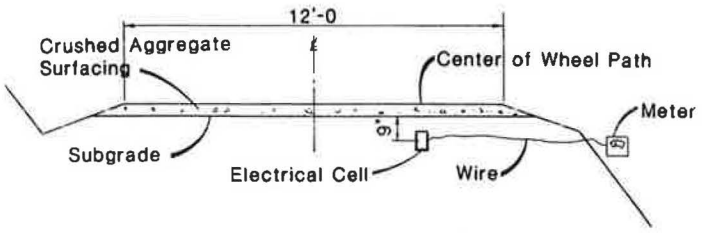

FIGURE 4 Electrical cell field installation.

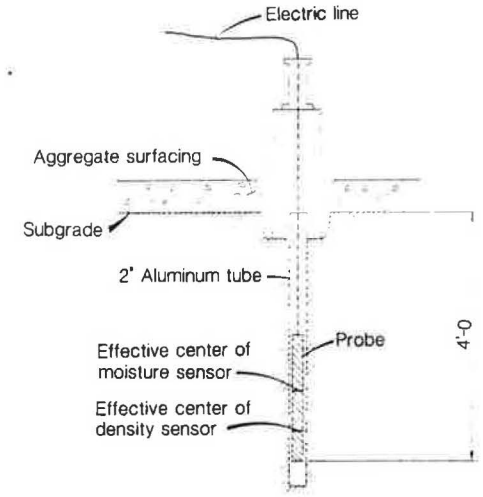

FIGURE 5 Nuclear depth probe schematic.

	Number of Pairs	*Mean Difference, Nuclear Minus Gravimetric/ Volumetric Measurements*	*Standard Error of Difference*	*Probability Level*
Percent saturation	10	−21.0	11.2	0.10
Density, lb/ft^3	10	7.4	6.3	0.27

The probability level indicates a low probability of difference between the nuclear and gravimetric measurements. In other words, there is a poor correlation.

RESULTS

Electrical Cell Measurements

The results of field electrical cell measurements are shown in Figures 6–14. Moisture contents are converted to percent saturation to provide uniformity between various soil properties. Each point on the graphs represents the average of the cell readings on a road for a particular date.

Moisture variations followed similar annual trends. In general, moisture contents decreased with increasing temperature and increased with precipitation. There was a lag of 1 to 2 weeks before rainfall affected the cells. The increase in percent saturation from precipitation was generally small and storms with less than about 0.2 in. of rain per day did not seem to affect cell moisture.

Subgrade soil temperature closely paralleled mean daily air temperature, and air temperature turned out to be a better prediction tool because it is more readily available. Subgrade soil temperature readings were particularly useful to determine if the soil was frozen.

Nuclear Depth Probes

The average and standard deviations of density and moisture content at the 18 nuclear depth probe stations at Horse Creek in 1980 are presented in Table 2. Data for other years both from Horse Creek and St. Maries are similar. The nuclear depth probe measurements were discontinued after 1980 because of apparent scatter in data and the results of field accuracy comparison tests.

The 0.5-ft moisture readings showed greater variations in time than the deeper readings, and they compare roughly with the electrical cell data.

The average density for all 0.5-ft readings is 120.1 lb/ft^3 compared to the average of all Washington densometer tests taken on top of the subgrade of 117.6 lb/ft^3. Higher nuclear densities were also observed in the accuracy comparison tests at various depths.

Subgrade soil densities were higher than anticipated. One reason is the in-place soil derived from metamorphic rocks that gradually grades into weathered bedrock. High densities would be expected in deeper cut sections. Traffic compaction may also have contributed to high densities.

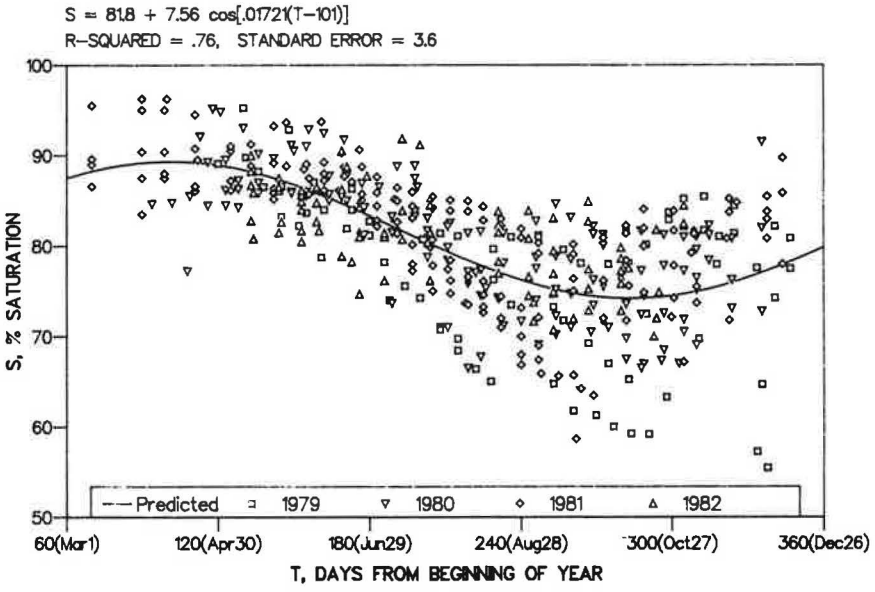

FIGURE 6 Total electrical cell data, cosine prediction.

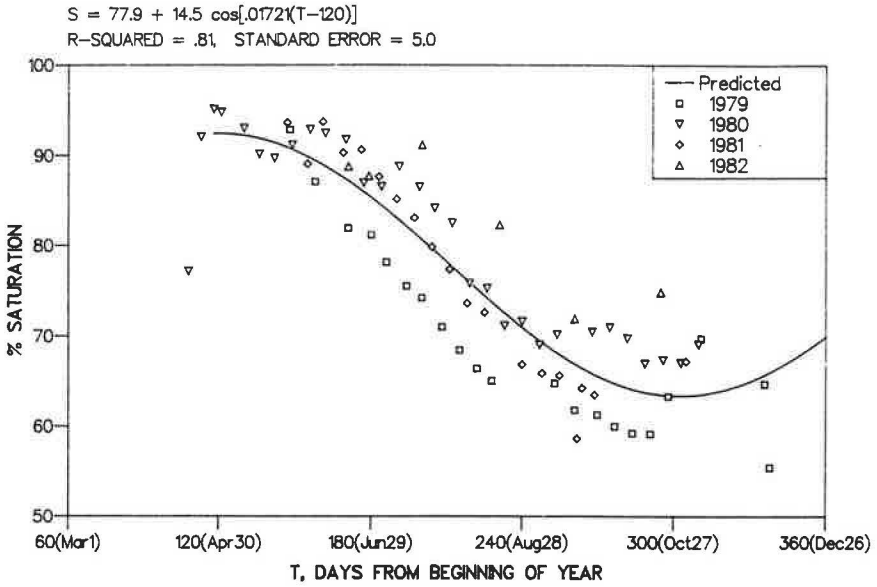

FIGURE 7 Horse Creek electrical cell data, cosine prediction.

ANALYSIS

Electrical Cell Percent Saturation

The electrical cell data were analyzed to find a practical model for prediction of percent saturation at various times during the year. The analysis was directed toward the period from spring thaw to snow closure in the late fall because these roads are normally closed during the winter months. The 1978 data were excluded from the analysis because of erratic data until the cells reached equilibrium with the surrounding soil. The following 21 variables were analyzed:

- Time;
- Precipitation;
- Cumulative precipitation;
- Antecedent precipitation index (defined below);
- Air temperature;
- Subgrade soil temperature;
- Aspect (facing direction of the topographic slope);
- In-place subgrade soil density;
- Maximum density, AASHTO T-99;
- Optimum moisture content, AASHTO T-99;
- Percent passing No. 4 sieve;
- Percent passing No. 200 sieve;
- Specific gravity;
- Unified soil classification;
- Type cross section such as through cut, through fill, or cut-fill combination;
- Inside or outside wheel track;

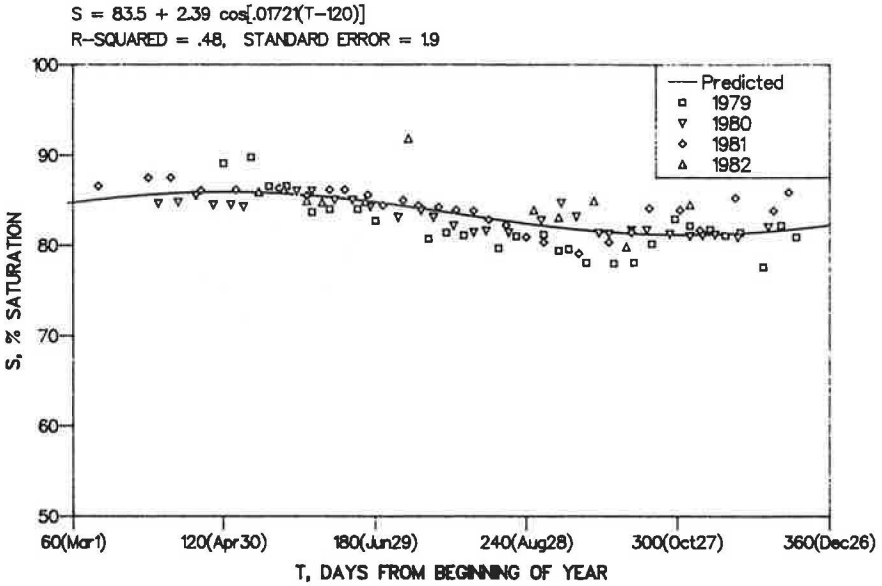

FIGURE 8 Staples electrical cell data, cosine prediction.

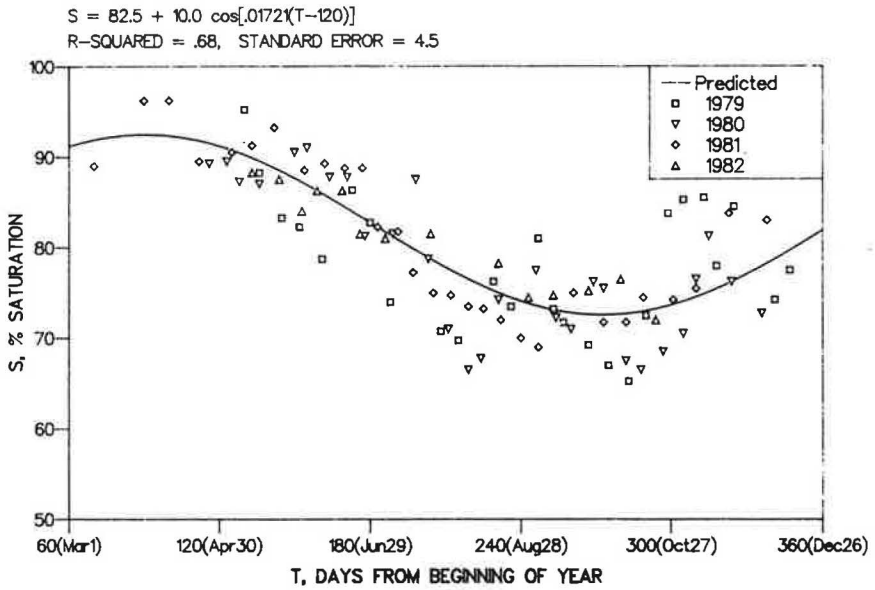

FIGURE 9 Xmas electrical cell data, cosine prediction.

- Side slope above road;
- Side slope below road;
- Longitudinal road grade;
- Depth of cut or fill; and
- Aggregate thickness.

Various transformations and combinations of variables were used.

Scatter plots, partial correlation analysis, and regression analyses, including nonlinear techniques, were made using the Number Cruncher Statistical System computer program (*2*) to identify significant variables and develop models. The best two models involved just one variable; either time or antecedent precipitation index. A number of functions involving precipitation, including a correction for the observed lag time between precipitation and soil moisture occurrence, were attempted. Attempts were also made to combine the time and precipitation functions, but the various mathematical combinations were no better than models with just one of the variables. None of the other variables or their transformations increased the R^2 value by more than 0.02, and the best total increase, using all significant variables, was less than 0.06. The most significant of the minor variables were air temperature, subgrade soil temperature, percent passing the No. 200 sieve, aspect, and elevation.

The data were analyzed by road for each year, by road using the data for all years, and for all data from all roads together. Once the minor variables were eliminated, the average percent saturation for each road on a particular date was used as the dependent variable.

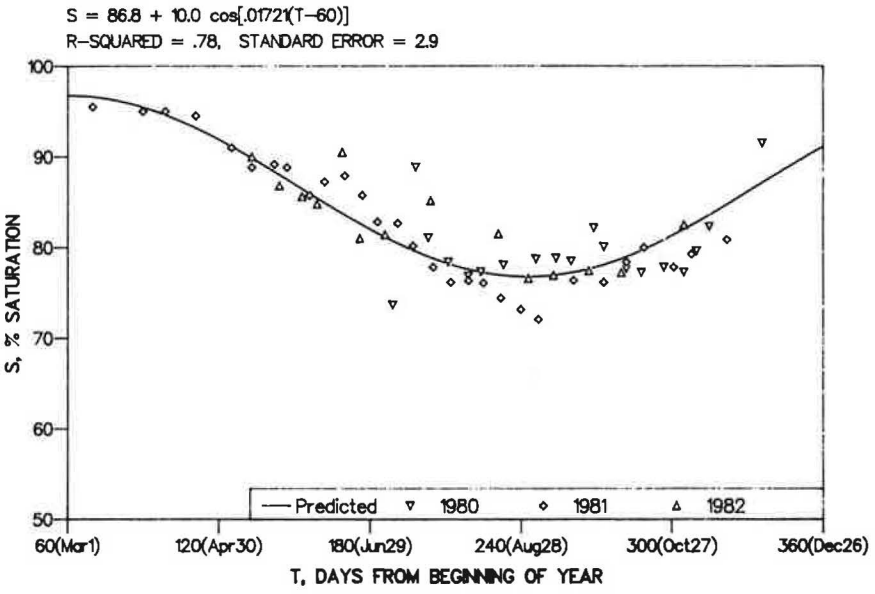

FIGURE 10 Merry electrical cell data, cosine prediction.

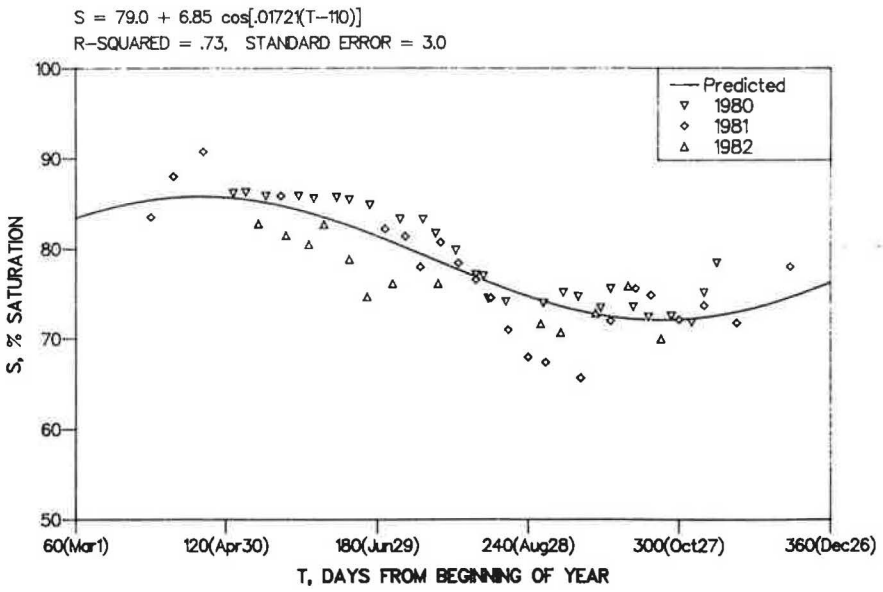

FIGURE 11 Gold electrical cell data, cosine prediction.

The best model found that involved time used a cosine function of time as the independent variable:

$$S = B + A \cos\left[\frac{\pi}{L}(T - T_0)\right] \quad (1)$$

where the quantity in brackets is expressed in radians and

S = percent saturation;
B = average percent saturation throughout the predicted period, (Max + Min)/2;
A = variation of percent saturation, (Max − Min)/2;
L = length of dry season (days), from spring thaw to the driest time during summer or fall;
T = time (days) from January 1 to the date of prediction; and
T_0 = time (days) from January 1 to the beginning of spring thaw.

Considering all data together, the best result was with L = 182.5 days (half a year), resulting in an annual cyclic function. The average date of spring thaw was Day 101 (April 11). A and B were solved by regression, resulting in the following equation:

$$S = 81.8 + 7.56 \cos [0.01721(T - 101)] \quad (2)$$

R^2 = 0.76, Standard Error = 3.6.

A plot of the prediction equation with the observed field data is shown in Figure 6. The results of similar analyses on each of the individual roads are shown in Figures 7–13.

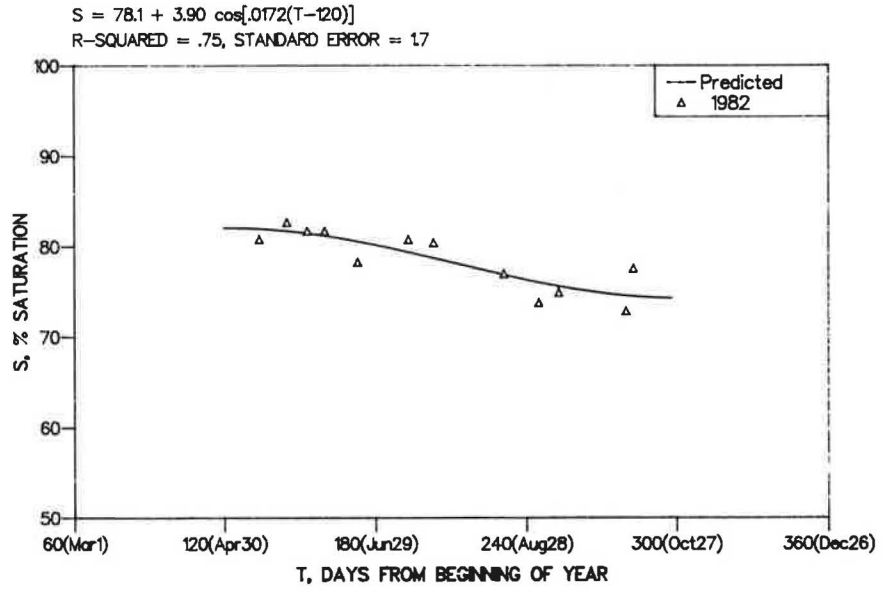

FIGURE 12 Emerald electrical cell data, cosine prediction.

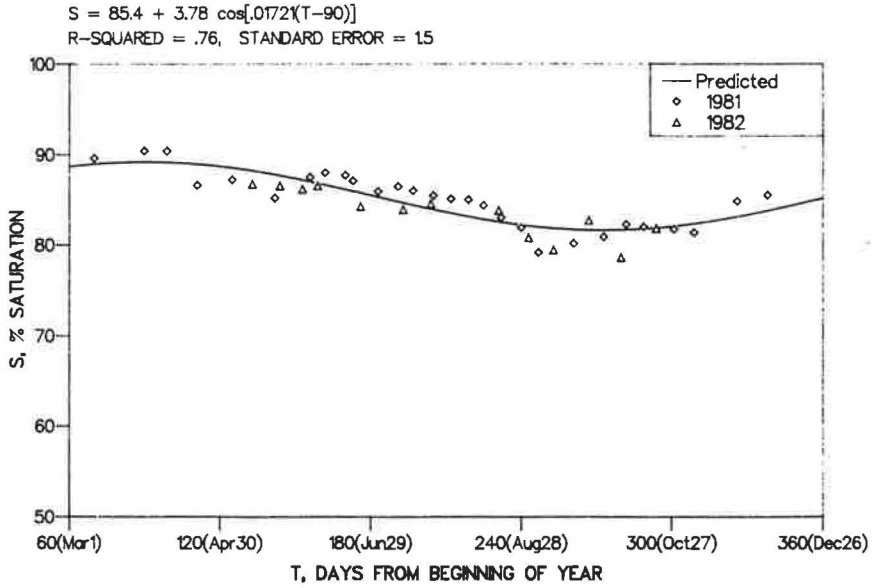

FIGURE 13 Elk electrical cell data, cosine prediction.

The preceding time model requires some knowledge of local conditions. The data shown in the figures should provide some guidance for selecting the parameters under similar site conditions.

The other successful model uses the antecedent precipitation index (API), that has been found useful in rainfall-runoff correlations to account for soil moisture (*3*).

A satisfactory relationship between percent saturation and API was developed for the Horse Creek data, but not for the St. Maries data. The reason appears to be the quality of precipitation data available. At Horse Creek, a weather station was available within 0.5 mi of the site. The nearest weather station for the St. Maries roads was the Clarkia Ranger Station, about 8 mi from the sites, which has an elevation difference of up to 2,100 ft. In mountainous areas, precipitation can vary drastically over short distances. Therefore, this model may only apply where accurate precipitation records are available close by.

The basic concept of API is illustrated by the formula

$$\mathrm{API}_n = K(\mathrm{API}_{n-1}) + P_n \tag{3}$$

where

API_n = antecedent precipitation index on the current date,
K = constant,
API_{n-1} = antecedent precipitation index on the previous date, and
P_n = precipitation occurring between Dates $n - 1$ and n.

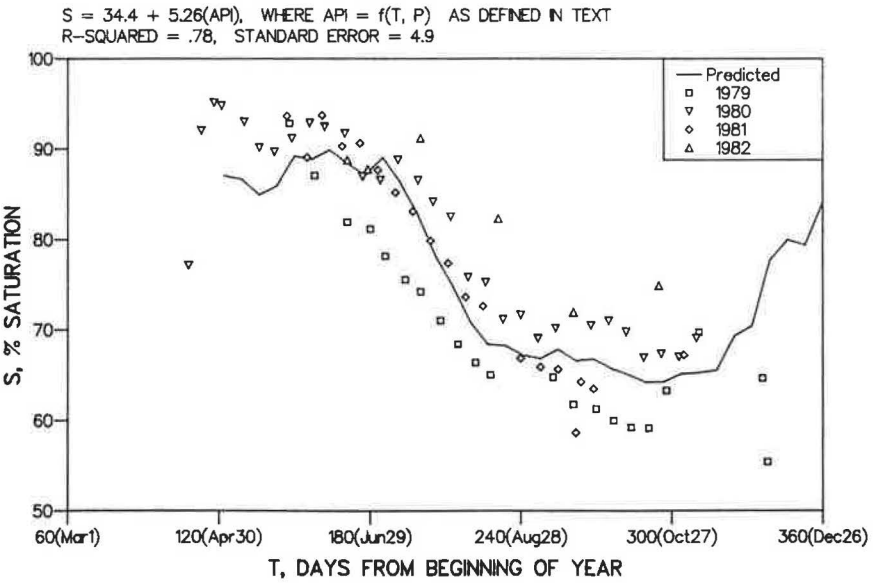

FIGURE 14 Horse Creek electrical cell data, API prediction.

TABLE 2 NUCLEAR DEPTH PROBE MOISTURE CONTENTS AND DRY DENSITIES, HORSE CREEK, 1980

Depth below Subgrade, ft.	DATE 5/4/80	5/27/80	7/3/80	8/3/80	8/31/80	9/28/80
0.5	14.2 (2.8) 116.7 (7.0)	13.7 (2.6) 119.2 (6.7	13.9 (3.6) 117.4 (9.7)	12.8 (2.8) 118.2 (9.8	13.1 (3.5) 122.0 (12.7)	11.7 (3.4) 122.1 (8.6)
1.0	14.7 (3.0) 116.4 (6.4)	14.5 (2.1 117.1 (5.9)	15.2 (3.3) 114.3 (8.4	14.0 (2.9) 118.4 (8.0)	13.1 (2.6) 121.1 (9.7)	13.6 (2.7) 116.0 (5.7)
1.5	14.0 (2.7) 117.7 (5.4)	14.5 (2.8) 116.1 (5.8)	14.2 (3.2) 117.3 (8.1)	13.9 (4.2) 116.5 (9.7)	13.2 (3.7) 120.1 (8.5)	13.3 (4.3) 116.5 (9.5)
2.0	14.5 (4.2) 116.2 (11.3)	14.3 (4.0) 117.4 (10.2)	14.2 (4.2) 118.0 (10.2)	13.9 (3.9) 118.8 (8.2)	13.4 (4.0) 118.9 (9.2)	13.6 (4.2) 116.8 (9.6)
2.5	14.6 (6.9) 117.1 (13.1)	14.5 (3.5) 116.1 (10.5)	14.3 (3.6) 117.1 (10.7)	13.4 (3.4) 120.4 (9.4)	13.0 (3.4) 122.5 (9.6)	13.2 (3.2) 116.9 (8.6)

Note: Top number in each set is % moisture content and lower number is dry density in pcf. Numbers in parentheses are standard deviations.

Various precipitation time periods, initial starting values of API, and K values were analyzed. The best correlations resulted from average weekly precipitation, API starting value = 10.0, K = 0.9, and time beginning at spring thaw. The weekly precipitation data were based on the average over a 5-year period. The calculation of API is tabulated below for the Horse Creek data.

Date	*Previous Week's API* × 0.9	*Previous Week's Precipitation (in.)*	*API, This Date*
May 2	–	–	10.00
May 9	9.00	0.92	9.92
May 16	8.93	0.66	9.59
May 23	8.63	1.15	9.78
May 30	8.80	1.60	10.40

The API regression equation for Horse Creek is

$$S = 34.4 + 5.26(\text{API}) \quad (4)$$

$R^2 = 0.78$, Standard Error = 4.9

where

S = percent saturation, and
API = weekly antecedent precipitation index.

A plot and observed data are shown in Figure 14.

Nuclear Depth Probe Data

Regression analyses were run on the nuclear depth probe data both with moisture and density as functions of depth, time, and the other variables analyzed with electrical cell data. The analyses were inconclusive and significant correlations were not found. The problem appears to be scatter in the data as discussed earlier.

CONCLUSIONS

Conclusions for the specific site conditions at the Horse Creek and St. Maries study areas were as follows:

- There are significant seasonal variations in subgrade moisture under aggregate surfacing that need to be considered in laboratory testing and thickness design.
- The accuracy of electrical resistance cells for measuring percent saturation is about 1 to 2 percent as compared to the gravimetric (oven-dry) method.
- Nuclear depth-probe percent saturation and in-place densities between 0.5 and 2.5 ft below subgrade indicate little change with depth or time, although the accuracy of nuclear depth-probe measurements in this study is poor.
- Soil test results and field moisture observations showed wide variations, indicating the need for large numbers of tests and observations to provide statistically meaningful results in a study of this kind.
- The degree of saturation of shallow subgrade soil, as measured by electrical resistance moisture cells, could be predicted from the cosine function of time given in Equation 1.
- When good weather station data are available close to the site, the degree of saturation could be predicted as a function of the antecedent precipitation index by Equation 4.

RECOMMENDATIONS

- Conduct additional studies in areas of different climates, topography, and soils to verify or modify the results given herein.
- Perform additional research to improve and develop instrumentation for monitoring subgrade moisture.

ACKNOWLEDGMENT

The work was accomplished by a team of USDA Forest Service and University of Idaho people. Appreciation is extended to Stephen Monlux and the Northern Region Materials Testing Laboratory personnel, Maridean Appell of St. Maries Materials Laboratory, James Hardcastle and his students at the University of Idaho for laboratory and field work, and Douglas McClelland for preliminary data analysis.

Support for the project was provided by the Forest Service Northern Region Engineering Unit and the Intermountain Forest and Range Experiment Station.

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New Variable Impact Test for Low-Volume Roads

G. KING, P. R. FLEMING, J. R. BOYCE, AND C. D. F. ROGERS

The development of a new apparatus for routine testing of subgrades and pavement foundations is described. Potential applications include pavement design, material selection, and proof testing during construction. The main objectives were to develop a relatively low-cost, lightweight, and portable dynamic testing apparatus that has the capability of producing on-site resilient modulus values. A prototype swinging hammer apparatus (ODIN) has been built and proving trials have been in progress for 12 months. ODIN is designed so that the stress and area of impact can be adjusted to simulate various loading conditions to suit the materials and constructions under test. Under low impact stresses, quasi-elastic (resilient) behavior occurs, followed by plastic deformation and failure at higher stresses. Digital storage of the impact pulse facilitates processing of the signal to determine the maximum resilient deflection. By using double integration techniques and incorporating energy balancing equations, a composite modulus value can be determined, taking into account both peak deceleration and pulse time. The apparatus typically produces an impact pulse of 1 to 3 msec duration, with a transient deflection of 0.5 to 3.0 mm. Intensive field testing of various materials from soft clay subgrades to capping and subbase layers has yielded reasonable moduli that correlate well with other tests. The ability to test different materials and constructions, along with the capability for data capture and interpretation, indicates widespread potential applications.

Design of low-volume roads needs to be innovative. Materials testing to determine parameters associated with pavement design, and subsequent proof testing of compacted layers, need to be efficient because of economic constraints. Routine testing of the subgrade and subbase is an important feature of pavement design and construction. However, the economics of low-volume roads imposes strict limitations on the testing regime.

It has been recognized, particularly for thinly surfaced roads, that determination of the resilient modulus of the pavement foundation is necessary to indicate whether a pavement will withstand repeated loading during trafficking. The relative complexity and expense in measuring a resilient modulus has, however, proved to be prohibitive for routine testing of low-volume roads. Despite attempts to calculate resilient moduli from California bearing ratio (CBR) values and combinations of standard laboratory test results, it is suggested that no acceptable method is in present use.

It is, therefore, considered that the development of a low-cost, lightweight, and portable dynamic testing apparatus for regular in situ determination of resilient modulus for unbound material layers has great potential for use in the design and construction of low-volume roads. The new variable testing apparatus, ODIN, has been developed by a teaching company formed between Geotechnics Limited and Loughborough University of Technology. It has been designed to test a range of natural soils, stabilized soils, and granular materials that are generally used in highway and earthworks construction. ODIN (see Figure 1), can be used for the initial determination of resilient modulus during design, for compliance testing during construction, and for the determination of local material suitability for use in construction.

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ODIN

ODIN consists of an impact head to which the interchangeable impact plates are secured. The head is raised by a swinging arm arrangement that has the advantage of allowing manual raising of masses up to 15 kg and that ensures a good contact between the impact plate and ground surface. Plates vary in diameter from 100 to 300 mm, allowing a range of impact stresses. The drop height can be varied up to 600 mm, with a stop facility to control drop height for simple repeated testing. An accelerometer mounted in the hammer head is used to measure deceleration on impact, with the signal being digitally recorded at 20-μsec intervals for a period of 5 msec from the moment of impact.

The impact stress can be controlled by variation of the diameter of the impact plate and the drop height. A stress range of 0.2 to 3.5 MPa can be achieved that simulates the stresses during wheel loading (typically 0.7 MPa for a single wheel load and standard axle), although the stress pulse is of much shorter duration. The stiffness of clays and granular materials is not generally dependent on rate of loading. However, it is recognized that the inertial stresses during testing can be significant and allowances are made during the analysis of results.

The diameter of the impact plate used during testing is important. Plate diameter will determine the depth to which a construction is stressed and influence the peak stresses acting during the test. It is also important that the area of the impact plate should be large enough to obtain a characteristic material response, without being overly influenced by individual aggregate particle size. For testing of homogeneous subgrade material, the larger plate sizes (200- and 300-mm diameters) have given consistent results at lower stress values. On subbase materials, both the smaller and larger plate sizes have been used with success. However, depending on the construc-

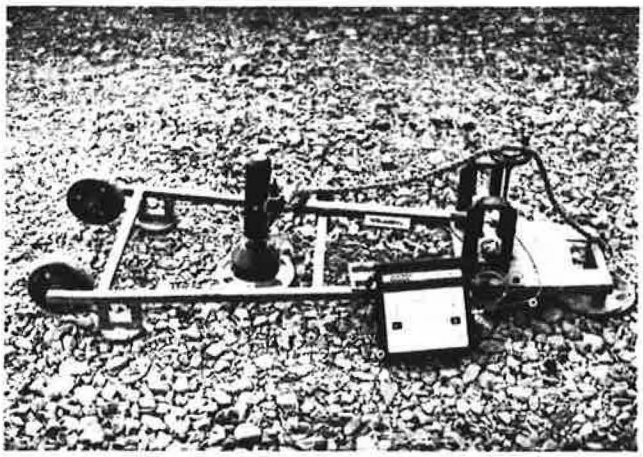

FIGURE 1 First ODIN prototype with data processor.

tion thickness and the plate sizes used, the surface modulus may be influenced by the subgrade stiffness.

The present test procedure involves testing at drop heights of 0.2, 0.3, 0.4, and 0.5 m, with generally four tests at each drop height. The accelerometer signals recorded at each height are generally repeatable, as shown in Figure 2, with the fourth drop used to calculate a resilient modulus, assuming repeatability. Note the straight-line loading and short transient loading pulse in Figure 2.

In a road construction environment, it is anticipated that full ODIN tests (i.e., testing at a number of drop heights and stresses) will be carried out at specified intervals to determine the elastic range of the material under test. More concise testing can then be carried out at much closer spacings from a specified drop height, allowing simple, quick, routine testing of subgrade and subbase.

The ability to vary the plate sizes allows ODIN testing to be designed for the particular materials and construction under test. With a small impact plate (100- and 150-mm diameters), testing of individual layers can be carried out, whereas larger plate sizes can be utilized to test either thicker homogeneous layers or to obtain a composite foundation modulus.

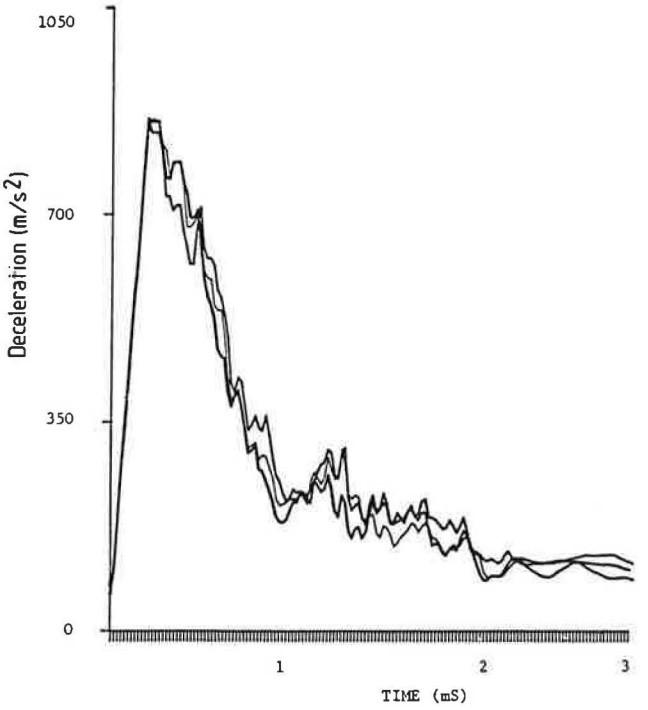

FIGURE 2 Accelerometer output for three successive tests demonstrating repeatability.

EXISTING APPARATUS

Existing methods of in situ pavement evaluation include static, wheel, and dynamic loading methods.

Static Loading

The in situ CBR test can be used to monitor individual layers, but is subject to increasing error with increased particle size. The CBR test is a bearing test that leads to shear failure and relates more to shear strength than resilient modulus.

The plate loading test involves the application of a load to a circular plate to obtain an elastic modulus (*1*). The duration of the loading is long compared to an actual wheel load. The large kentledge required and the time taken to carry out a single test restricts its potential for routine testing.

Moving Wheel Loading

Deflection beam testing is used to measure pavement deflection as a wheel load passes, but is often unreliable on granular materials because of significant movement during loading.

Dynamic Loading

The falling weight deflectometer (FWD) test involves a falling weight (of up to 400 kg) the impact of which is transmitted to a circular loading plate by a system of springs, giving a pulse time similar to that of vehicle loading. A series of geophones measures ground deflection at an increasing radial distance from the loading plate. Computer analysis techniques are used to calculate the resilient moduli of the separate pavement layers. The equipment and associated computer software are expensive, and hence, regular testing is not feasible in low-volume road situations.

The Clegg hammer apparatus (*2*) consists of a 4.5-kg mass, fitted with an accelerometer. The output from the accelerometer is in the form of a Clegg impact value (CIV). The hammer is dropped down a guide tube from a height of 450 mm four times, the CIV being the fourth or highest deceleration (1 CIV = 100 m/sec^2). The impact area is 50 mm in diameter and is similar in size to the coarser subbase aggregate particles. As a result of this small impact area, high stresses occur that cause localized bearing failure in all but the stiffest construction materials. It has been shown (*3*) that the CIV, which is overly influenced by properties of material close to the loaded area, is more likely to reflect the shear strength or CBR of the material rather than its resilient modulus.

THEORY AND ANALYSIS

The theory that has evolved during the development of ODIN is that under low impact stresses quasi-elastic (resilient) behavior occurs, followed by plastic deformation and failure at higher stresses (*3*). For a homogeneous material, the factors that influence deceleration on impact are the soil stiffness, the bearing capacity of the material under test, and inertia effects.

1. Stiffness. Laboratory testing (*4*) has indicated that resilient modulus is stress dependent, although not dependent on rate of loading, in the time range associated with transient vehicle loading of pavements.
2. Bearing Capacity. In situ testing with high stresses, as is the case with the Clegg hammer, can produce a bearing failure resulting in significant plastic deformation or compaction.
3. Inertia. Inertia effects are inherent in impact testing. The loading pulse produced by ODIN is rapid, of the order of 1 to 3 msec, and energy is required to accelerate the soil mass to the same velocity as the impacting plate. This inertial effect is allowed for in the calculation of the apparent modulus on impact (*5*).

By equating the impact energy to the work done in deflecting the ground under the impact area, relationships between peak deceleration (A_p), maximum deflection (s), drop height (H), and resilient modulus (E_r) can be derived.

Digital storage of the impact pulse facilitates simple signal processing. A typical unfiltered signal is shown in Figure 3 together with a filtered signal using a 4-kHz digital filter. Peak deceleration is obtained from the filtered signal, whereas velocity and peak displacement are obtained by integration (once and twice, respectively) of the unfiltered signal. In order to minimize inertia effects, peak displacement is used rather than displacement at maximum deceleration (*3,6*).

Using the relation that force is equal to mass multiplied by acceleration and assuming a uniform stress under the plate, maximum stress on impact (σ) is calculated from the peak deceleration value (A_p) as follows:

$$\sigma = \frac{4A_pMg}{\pi D^2}$$

where

M = mass of falling weight,
D = impact plate diameter, and
g = acceleration caused by gravity.

Resilient modulus (E_r) is calculated using a Boussinesq approach:

$$E_r = \frac{A_pM(1 - \nu_r^2)g}{sD}$$

where

ν_r = resilient Poisson's ratio, and
s = peak displacement.

Assuming a value for Poisson's ratio, 0.3 for a granular material and 0.5 for a clay, the resilient modulus can be calculated from the accelerometer output.

The calculated modulus is normalized to a stress of 0.7 MPa to relate to typical stresses acting on the foundation surface materials from construction traffic. Stress correction is made using the following relationship (*4*).

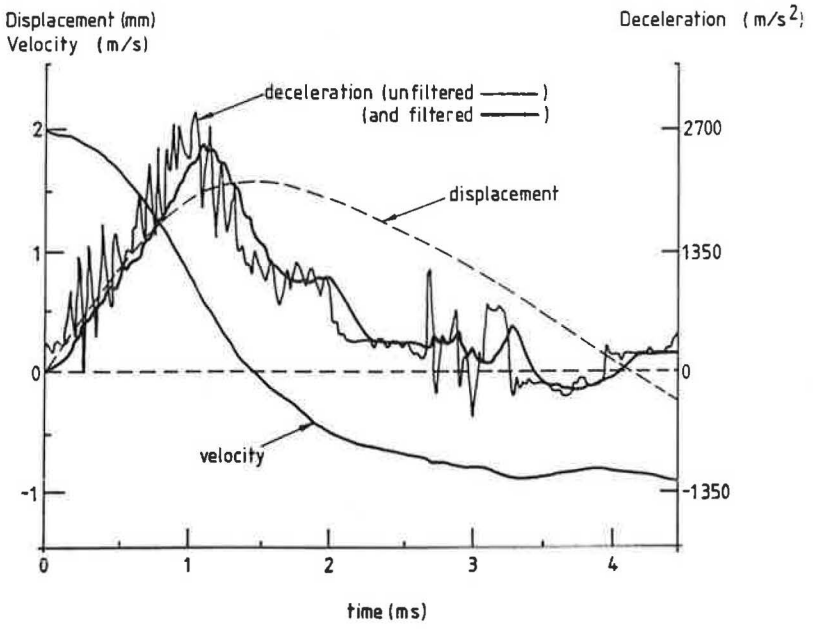

FIGURE 3 Typical unfiltered, filtered, and integrated accelerometer signal on impact.

$$E_r = k_1\theta^{k_2}$$

where

k_1 = material constant,
k_2 = material constant, and
θ = sum of principal stresses.

The range of k_2 values is generally between 0.2 and 0.7; hence, assuming a value of k_2 of 0.5,

$$E \propto \theta^{1/2}$$

Moduli values are only calculated from tests demonstrated to be within the elastic range of the soil under test. Figure 4 shows peak deceleration (A_p) versus drop height (H) for impact tests carried out both using 100- and 50-mm plate sizes. The larger (100-mm) diameter plate size indicates A_p is proportional to H at A_p values of up to about 2500 m/sec^2, with a leveling or reduction in value above this stress level indicating possible bearing failure. The smaller 50-mm plate and Clegg hammer give peak deceleration (A_p) values that increase slightly with increased drop height. This is suggested to be a result of high impact stresses creating a failure condition at low drop heights, with only minor increases in impact resistance for increasing heights thereafter.

Because of inertia effects, however, an increase in contact stress is to be expected with increased drop height. Variations of peak decelerations alone, therefore, may not indicate failure conditions in all tests. Deflections recorded during testing will also increase with increased contact stress, although a significant increase in deflection will occur when the yield and failure conditions are reached. Variations in the moduli (uncorrected for stress) with increased drop height best indicate whether the elastic range of the soil has been exceeded, this condition being characterized by a limit or reduction in calculated moduli values.

RESULTS

ODIN has been specifically developed for use on variable subgrades and differing unbound pavement and earthwork materials. Attention has been given to correlation of values obtained by ODIN to values obtained by other in-situ testing apparatus, specifically the plate bearing test, the FWD, and the Clegg hammer.

Field testing at Bothkennar, Scotland, undertaken in connection with research work being carried out by the University of Nottingham, provided the potential for correlation between ODIN, FWD, and Clegg hammer for 12 pavement foundation constructions. Nine of the pavement foundations consisted of crushed rock aggregate (Type 1), having the same material and grading, and three consisted of sand and gravel. A 30-m length of subgrade exposed during construction was also tested with the Clegg hammer and ODIN apparatus only. The pavement foundations varied in thickness and many were reinforced, with different reinforcements being used for different constructions.

Testing of the thick, homogeneous subgrade was carried out using larger plate diameters to minimize the stress on impact. Moduli values of the order of 25 to 45 MPa were obtained and considered to be reasonable. A slight reduction in stiffness with increased chainage can be identified from Figure 5. At the time of testing, the subgrade at 70-m chainage had only recently been exposed to relatively hot conditions, whereas that at lower chainages had been exposed for some time, allowing the development of a thick drier crust reflected in slightly higher modulus values.

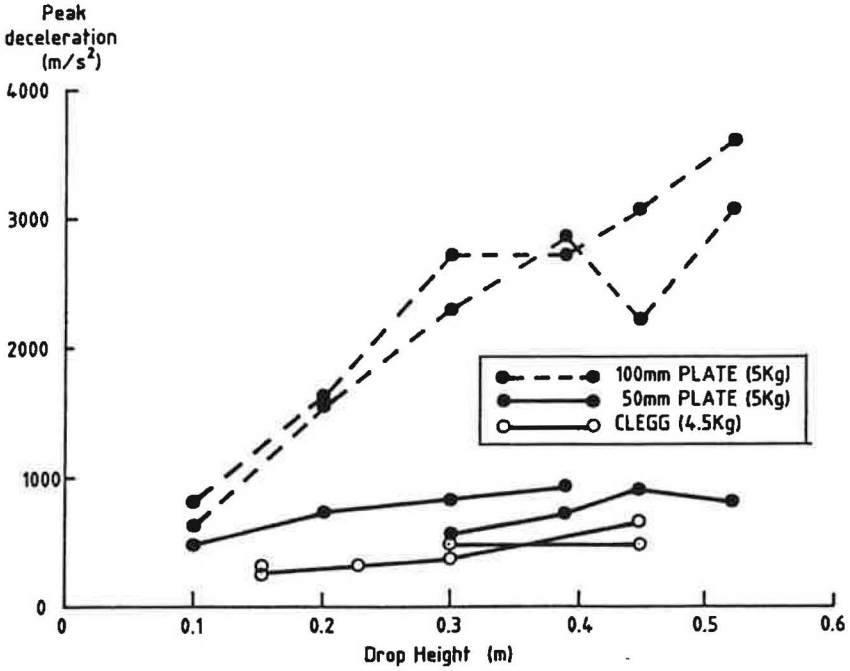

FIGURE 4 Relationship between peak deceleration and drop height for varying impact plate sizes.

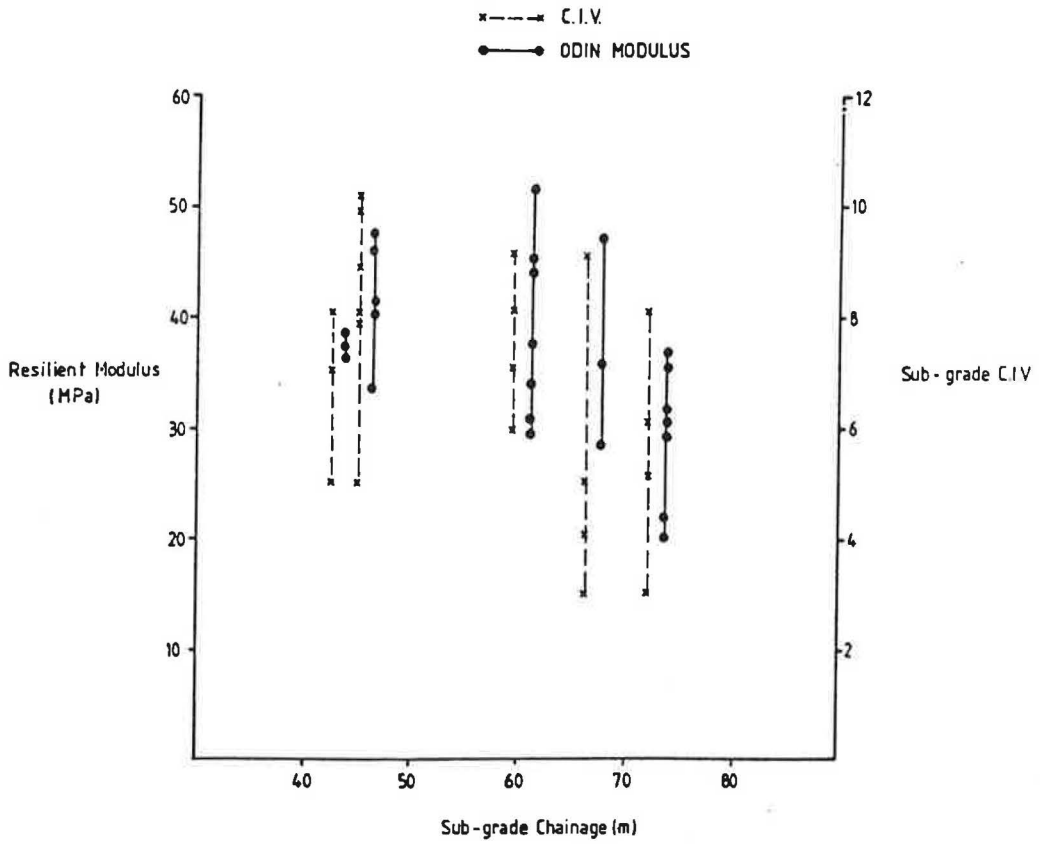

FIGURE 5 Variation of resilient modulus and CIV with increased chainage on clay subgrade.

Variations in CIV values indicate a similar trend (Figure 5); however, they are considered to reflect variations in CBR (or shear strength) rather than resilient modulus.

ODIN testing of the pavement foundations was carried out with smaller diameter plate sizes to control the depth of stressing. FWD testing was carried out using a 300-mm plate, with the stiffness of the subbase layer back-calculated for a normalized stress of 500 kPa. The respective back-calculated FWD subbase moduli and ODIN moduli obtained with a 200-mm plate are shown in Figure 6. It can be seen that a reasonable correlation exists between the FWD and ODIN moduli, although it is evident that specific correlations exist for particular material types. Within any test foundation a range of moduli was obtained with both the FWD and ODIN, and it is considered that this reflects the true variability of the granular constructions. No correlation was obtained between CIV and either FWD or ODIN moduli.

A detailed program of field testing was carried out on pavement foundation test bays at the Road and Railroad Research Department of Delft University, Holland. The constructions tested consisted of seven unsurfaced pavement foundations comprising 250 mm of subbase overlying a sand subgrade. The subbase materials varied from weak lava to stiff self-cementing slag materials. Testing was carried out using the ODIN apparatus and the Clegg hammer. Earlier work carried out at Delft included testing with a modified FWD (*7*). This dynamic plate bearing test measures only the central deflection, hence giving a composite modulus. Figure 7 shows the composite modulus values both for ODIN using a 150-mm impact plate and the FWD using a 300-mm loading plate for the various constructions tested, along with the CIV.

The moduli are generally comparable, although both tests show some scatter in particular bays. This relation is further demonstrated in Figure 8, which shows the correlation of composite modulus for ODIN and the FWD. Some scatter is inevitable when testing granular materials because of such features as surface texture and variations in density. When a significant range of moduli values was obtained with ODIN, as in the case for the phosphorous slag (FO), a comparable range of values is evident for the modified FWD.

No apparent correlation was found between surface modulus and CIV for the constructions. It is suggested that the Clegg hammer has differentiated between constructions with either loose surfaces or weak constituent aggregate particles and those constructions with a closed texture or high aggregate particle strength.

APPLICATIONS

In the United Kingdom and many other countries, the construction of formation, capping, and subbase layers relies on material gradings and method specifications for their placement and compaction. It has been recognized for some years that it is desirable to move towards a situation where the ability of the pavement foundation to support the pavement

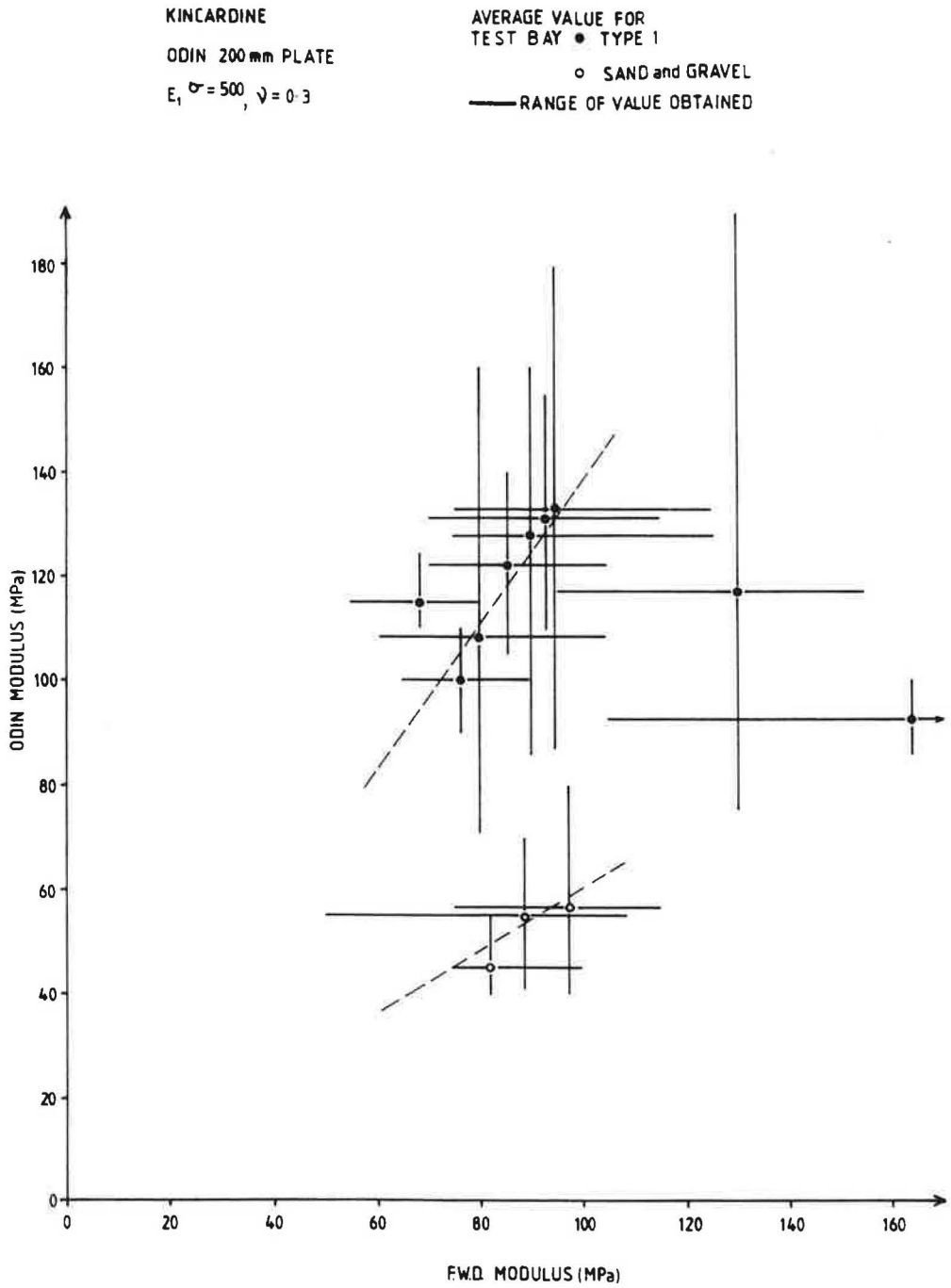

FIGURE 6 Resilient modulus and FWD back-calculated modulus for 12 subbase constructions.

structure is assessed more directly. Empirical tests such as the CBR, Moisture Condition Value (MCV), and CIV may give some assurance that the material is well compacted and unlikely to suffer permanent deformation, but they cannot show that the surface modulus is adequate to support a pavement subject to repeated loading.

It is believed that applications will be in two stages. The first stage is the testing of materials before a road contract commences. The second stage includes routine testing of subgrade and foundation layers as constructed to ensure that a consistent performance is achieved throughout the contract.

CONCLUSIONS

The new impact test apparatus is considered to have great potential for use in low-volume road situations. The low cost, robustness, and portability of ODIN, along with the ability to produce on-site resilient modulus values, enable use both for routine testing during the design and construction of pavements and the determination of material suitability for use in construction. The ability to vary the impact plate size and drop height also allows ODIN testing to be designed for the particular materials and construction under test. With appro-

OS Eastern Scheldt Sand (same as subgrade).
MG Crushed masonry.
FF 50% crushed Concrete/50% crushed masonry.
LA Lava.
SS Pelletised blast furnace slag.
ME Crushed masonry with 15% electro-furnace slag
FO Phosphorous slag.

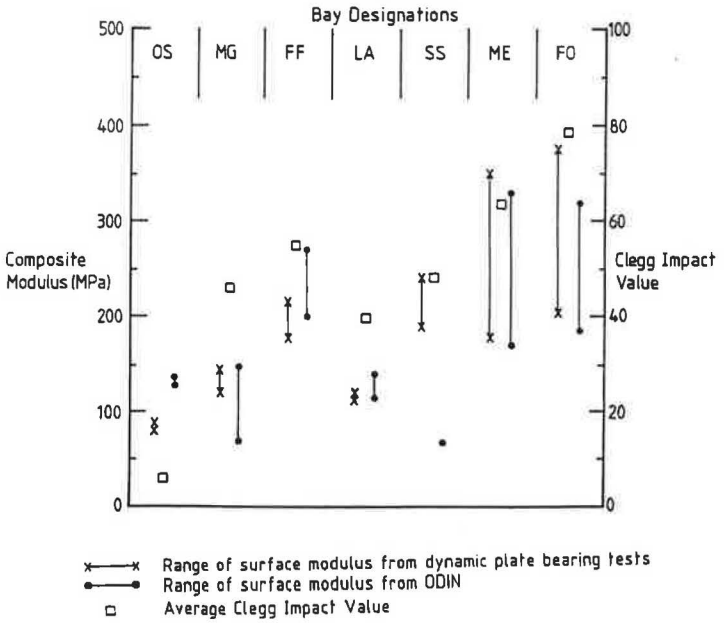

FIGURE 7 Composite modulus (ODIN and FWD) and CIV for seven construction bays.

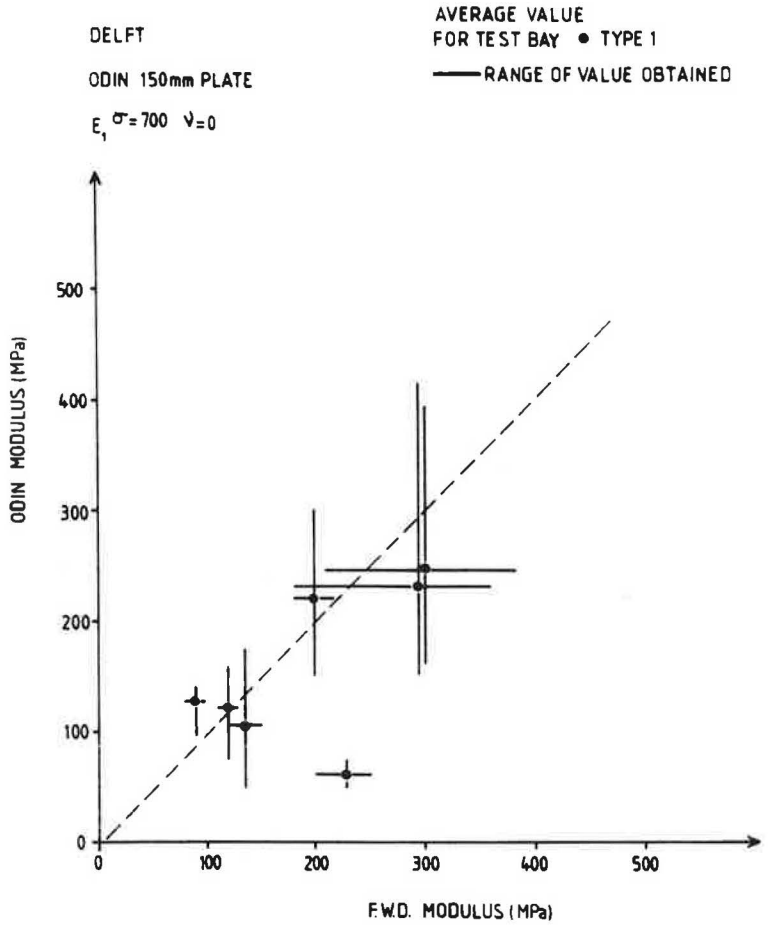

FIGURE 8 FWD and ODIN modulus values for bays tested.

priate analysis of the impact pulse, a modulus can be determined for a wide variety of soils and unbound materials.

By equating the impact energy to the work done in deflecting the ground under the impact area, relationships have been derived to calculate a resilient modulus. ODIN testing produces a repeatable and effective means of determining resilient modulus, which compare favorably with those values obtained by the FWD over a range of stiffnesses. It has also been suggested that although the Clegg hammer is often correlated with CBR, being a parameter more indicative of shear resistance it has no correlation with elastic moduli.

A second prototype is now under construction. This model will be used to further examine the sensitivity of the results to compaction, moisture content, and other construction variables, and refine analysis procedures. Current field test programs include specifically the study of correlations with other in-situ foundation evaluation procedures, both static and dynamic.

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Expert System for Hazard and Risk Assessment on Low-Volume Hill Roads in Nepal

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An expert system that is at the stage of a developmental prototype is described with an emphasis on how it increases the applicability and reliability of hazard and risk assessment on low-volume hill roads in Nepal. The informal and ill-structured knowledge used for hazard and risk assessment is shown to be organized and standardized using a knowledge-based expert system. The use of site- and failure-specific diagnostic procedures is shown to be a significant improvement over current methods of hazard and risk assessment that make use of aggregate weights only. A sample user consultation demonstrates the operation of the expert system.

Hill roads in Nepal service subsistence economies. Although the low level of economic activity results in low traffic volumes, the geologically active mountain chain introduces risks to the road that lead to high costs. Thus, hill roads in Nepal are low-volume, high-cost roads. Table 1 indicates this point.

Natural hazards such as rockslides and slumps pose risks of losing investments in hill roads. In order to plan, design, and construct a hill road that accounts for hazards in the form of tolerable risks, it is necessary to incorporate a systematic process of hazard and risk assessment into the practice of road construction. Hazard and risk assessment requires the use of extensive judgment to identify potential dangers, the likelihood of experiencing a danger in a given time (the hazard), the vulnerability of engineering structures on different standards of roads to damages from natural or man-induced hazards, and the changes in hazard as a result of countermeasures taken to reduce the expected value of loss (the risk). The practice of hazard and risk assessment was introduced only recently in Nepal, because the collection of judgments needed exists only as an ill-structured body of knowledge that is based on specific experiences. To extend this ill-structured knowledge into the practice of hill road construction on a national scale, there is a need to produce a standardized approach to hazard and risk assessment that can be readily transferred to practicing engineers and engineering geologists.

Knowledge-based expert systems (KBESs) provide a robust problem-solving environment to formally organize ill-structured knowledge and to produce a practical tool for carrying out intelligent tasks performed by highly skilled people. Given the subjective, incomplete, ill-defined, and informal collection of hazard and risk assessment knowledge currently existing in Nepal, a convenient way to standardize the hazard and risk assessment procedure for hill roads is to develop a KBES. A KBES will enable a widespread incorporation of hazard and risk assessments into the practice of hill road construction.

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KBES IMPROVEMENTS TO HAZARD AND RISK ASSESSMENT APPROACH

A KBES demonstration prototype was developed with the intent to produce a systematic procedure to hazard and risk assessment while developing a standardized technique. It is believed that the development of a systematic procedure, and specially a computerized one, will enable nonspecialists to address hazards and risks conveniently. Also, the standardization will result in an improved reliability of hazard and risk assessment results.

Existing hazard assessment techniques in Nepal are based on a system of relative weights that express the contribution of natural factors to a potential instability (*1*). For example, in the case of hazard assessment during the preliminary stage of a road project, all sections of the proposed alignment are assigned a set of ratings depending on the presence or extent of (a) slope attributes—angle, relief, and complexity; (b) climate—annual rainfall, antecedent moisture conditions, and probability of cloudburst; (c) geology—rock type, discontinuities, soil type, and soil thickness; (d) land use—vegetation, roads, and canals; (e) geodynamic processes—existing mass movements, stream undercutting, glacier- or landslide-dammed lakes; and (f) faulting and seismicity. The ratings are then summed along the entire alignment and divided by the length of the alignment to arrive at the relative hazard level per kilometer. A similar approach is taken during hazard assessment at the feasibility study stage (*1*). Weights are assigned to factors that contribute to rock and debris slides. A given site may be rated as high, medium, or low hazard depending on the presence of factors that reflect the structural, lithological, hydrogeological, and weathering conditions. A computerized version of this approach has been developed as well (*2*). The ratings or weights are based on field experiences in Nepal (*3*).

The existing approach to risk assessment is oriented towards the evaluation of road length that will most probably be lost because of damages from hazards over the road life (*1*). This approach is designed for use during the preliminary stage of road construction to aid in alignment selection and involves risk calculations on successive sections of the alignment facing

TABLE 1 HILL ROAD COSTS IN NEPAL

Road	Length (km)	Road Standard	Traffic (1990 AADT)	Average Cost per Kilometer (1990 U.S. $)
Dharan-Dhankuta	50	Double-lane, paved	197	797,000
Mugling-Narayangat	36	Double-lane, paved	852	445,000
Dhangari-Dandeldhura	135	Double-lane, dirt	<50	246,500
Kohalpur-Surkhet	92	Double-lane, graveled	<50	498,000
Lamahi-Tulsipur	24	Single-lane, graveled	131	181,500
Lamosangu-Jiri	110	Single-lane, paved	138	214,300

different types of hazard. Risk assessments during latter stages of road construction are treated with general indications only (*1*).

Expert systems have been developed for problems in other areas of geotechnical engineering. CONE (*4*) classifies soils and infers the shear strength from cone penetrometer data; RETWALL (*5*) addresses the problem of choosing applicable retaining wall types for different conditions; Shallow Trench (*6*) aids in interpreting a new soil classification system to plan safety precautions during excavation for shallow trenches; and SOILCON (*7*) provides advice on how much subsurface investigation is needed given what is known and the requirements of a proposed structure. These expert systems are either at the stage of a development or operational prototype (*8*).

The KBES demonstration prototype differs from the rating method of hazard assessment in that the KBES diagnoses different types of failures using specific rules along the alignment, whereas the rating method arrives at an aggregated hazard level per kilometer through a black-box approach. In fact, a major difference between the KBES and present approaches to hazard assessment (both the rating and relative weight methods) is that the former gives explicit justifications for its results, whereas the latter produce results containing implicit judgments that are not amenable to review. Thus, whereas the KBES can be updated or used with unique or new considerations in a systematic way, the current methods are too rigid to allow convenient modifications. The computerized approach to hazard assessment, being algorithmic, only automates the existing manual method. It does not improve the method of hazard assessment itself as the KBES does. Also, although the results of hazard assessment using the KBES are designed to fit into the procedure for risk assessment, the previous hazard assessment techniques cannot be used for risk assessment because forecasting with respect to potential damage to road elements is not included. The KBES is a significant advance over the current methods of risk assessment in that it also introduces currently nonexistent rules to assess the vulnerability of different structures facing different hazards and to modify the likelihood of experiencing a danger given the road design to be implemented and different countermeasures.

The KBES is an addition to the list of existing expert systems for geotechnical engineering. This KBES is most similar to CONE and Shallow Trench, in that heuristics prevalent in the practice of geotechnical engineering are systematized by all three. This KBES, like CONE, is built with a general-purpose representational language, whereas SOILCON, Shallow Trench, and RETWALL use expert system shells. Unlike CONE, this KBES runs on a personal computer.

KBES STRUCTURE AND USE

The KBES has been implemented in a version of PROLOG called Turbo PROLOG (*9*). PROLOG has been classified as a general-purpose representation language that uses a logic-based representation of knowledge and a backward-chaining inference engine (*10*). Turbo PROLOG was chosen because the backward-chaining control strategy is suited to the diagnostic nature of the hazard and risk assessment procedure, has complete flexibility, and runs on a personal computer. Turbo PROLOG allows calls to algorithmic procedures written in Turbo Pascal.

The context is described to the KBES by the user. Beginning at chainage location zero, the KBES queries the user on various hazard and risk parameters on user-defined segments that may vary according to homogeneous units of data. The query for a given parameter begins at the least-detailed level and proceeds to higher levels of detail until the user's information is exhausted. For example, in acquiring information regarding terrain composition on a given road section, the KBES starts out by asking the user if the area is composed of resistant or nonresistant rocks. If the user replies resistant, the KBES asks whether the rocky terrain is interbedded or not, what the rock type is, and what the orientation of the rock structure was with respect to the proposed road alignment. Also, the KBES acquires information regarding such factors as drainage pattern, existing landslides, geology, vegetation, slope, seismicity, glaciation, road standard, and proposed structures. The smallest unit that can be described by the user is 100 m. Depending on the scale of topographic map or air photographs, higher units may be used for preliminary studies. The entire alignment is covered and the information is preserved in a data base.

The hazard diagnosis knowledge base exists as a series of rules that define the conditions sufficient to produce any of the common mass movements. There may be more than one rule that defines a particular mass movement; such multiple rules are used to account for the different levels of detail contained in the information the user provides. The mass movements diagnosed by KBES are as follows:

Soils	*Debris*	*Rock*
Slump	Debris flow	Planar slide
Translational slide	Debris slide	Wedge slide
Mudflow		Toppling
Surface erosion		Rockfall

Each hazard diagnosis rule has an attached probability of realizing that danger in a given time interval. The probability of one failure during the time interval (of the road life) is

TABLE 2 CONTEXT OF SAMPLE USER CONSULTATION

Slope	
Chainage	Degree
7.0-7.8	36-45°
7.8-8.0	45-50°
8.0-9.1	36-45°
9.1-9.2	46-50°
9.2-9.4	>51°
9.4-10.0	46-50°

Rock Type	
Chainage	Type
7.0-7.8	Fractured slate
7.8-8.2	Colluvium
8.2-8.6	Slate
8.6-8.8	Colluvium
8.9-9.0	Interbedded quartzite + claystone
9.0-9.1	Eluvium
9.1-9.3	Interbedded quartzite + shale
9.3-9.4	Colluvium
9.4-9.9	Interbedded quartzite + shale
9.9-10.0	Thick-bedded quartzite

Vegetation	
Chainage	Type
7.0-9.0	Sparse brush-es
9.0-10.0	Sparse trees

Landslides	
Chainage	Type
7.1-7.2	Medium slump
7.2-7.3	Medium planar rockslide
8.1-8.2	Two large slumps and one small soilslide
8.8-8.9	A small slump and a large planar rockslide Debris flow
9.7-9.8	Two medium slumps
9.3-9.4	Two medium wedge failures
9.4-9.5	A large planar rockslide
9.9-10.0	

Drainage	
Chainage	Type
7.0-9.0	Dendritic
9.0-10.0	Trellis

Geologic Structure	
Chainage	Structure
7.0-8.0	Crushed and fractured rock
7.8-9.9	Folded area
8.2-8.5	Folded area
9.2-9.4	Crushed rock

Landuse	
Chainage	Type
7.0-10.0	Non - cultivated

Groundwater	
Chainage	Condition
7.0-7.5	Dry
7.5-8.0	Perennial seepage
8.0-8.4	Dry
8.4-8.5	perennial seepage
8.5-8.9	Dry
8.9-9.1	Perennial seepage
9.1-9.5	Dry
9.5-9.7	Perennial seepage
9.7-9.9	Dry
9.9-10.0	Seasonal seepage

assigned to the diagnostic rules using methods such as the comparison of lotteries (*11*). Also, each rule includes the extent of the hazard in the diagnosis as small, medium, large, and very large, depending on whether the length of the hazard along the slope or the alignment is between 3 and 10 m, 10 and 30 m, 30 and 100 m, or larger than 100 m, respectively.

The user may specify one of three possible road standards on the proposed alignment during the context definition. The standards are (a) low cost—cut slopes at marginal equilibrium with no mitigations used to control hazards, (b) medium cost—equilibrium cut slopes with partial mitigations, and (c) high cost—equilibrium cut slopes with full mitigation measures taken. For each standard, the vulnerability is defined by rules that estimate the likely road element losses on a given section subject to a given hazard in terms of an equivalent percent of road length. Risk is estimated by first using rules to modify the recurrence hazard as needed and then as a product of hazard, vulnerability, and net monetary worth appropriate to the first occurrence or subsequent recurrences over the road life.

The hazard and risk diagnoses are made on each 100-m interval along the entire alignment. On completion of these diagnoses, the KBES combines adjacent sections of 100 m that have the same hazard and risk diagnosis. This is the final result that is presented to the user. The user may ask for an explanation of why a particular section has the resulting diagnosis. The KBES will provide an explanation in terms of the rules that were used to arrive at the diagnosis in question.

Sample User Consultation Session

A small section of the Tulsipur-Salyan road in midwestern Nepal was presented to the KBES for hazard and risk assessment. The hazard-related information given to the KBES from 7.0 to 10.0 km is presented in Table 2. A low-cost road standard was specified with a cost of U.S. $140,000 per kilometer.

Using various hazard diagnosis rules, the KBES made the hazard diagnosis presented in Table 3. The 100-m unit that was subject to the same hazard has been combined by the KBES so that Table 3 indicates chainage sections subject to different hazards. The risk diagnosis results indicated in Table 4 indicate the most likely amount of loss in terms of monetary value caused by one occurrence of a given hazard sometime during the 20-year road life. Two examples of explanations given for the hazard diagnosis results are presented in Table 5 and interpretations of the explanation may also be requested by the user. The explanations shown in Table 5 indicate the reasoning followed by the KBES. The diagnosis was made because various parameters had the values presented in Table 5. The total risk for the 3-km section, because of the certain occurrence of each hazard at least once over the 20-year life, is U.S. $20,260. The sample run can easily be extended to the entire alignment to arrive at total alignment risk.

TABLE 3 HAZARD DIAGNOSIS RESULTS

Chainage (km)	Hazard Type	Probability of One Failure During Road Life (20 years)	Extent of Failure
7.0 to 7.3	Plane failure (rock)	0.5	Small
7.3 to 7.7	Wedge failure	0.9	Medium
7.9 to 8.2	Slump	0.9	Medium
8.2 to 8.4	Wedge failure	0.7	Small
8.8 to 9.0	Plane failure (rock)	0.9	Large
9.2 to 9.3	Wedge failure	0.6	Small
9.3 to 9.4	Slump	0.7	Small
9.4 to 9.5	Wedge failure	0.9	Medium
9.5 to 9.6	Wedge failure	0.7	Medium
9.6 to 9.7	Wedge failure	0.9	Medium
9.8 to 10.00	Plane failure (rock)	0.9	Large

TABLE 4 RISK DIAGNOSIS RESULTS

Chainage (km)	Vulnerability (% road length)	Risk (U.S. $)
7.0 to 7.3	2.5	524
7.3 to 7.7	2.0	1,006
7.9 to 8.2	3.3	1,248
8.2 to 8.4	2.5	490
8.8 to 9.0	25.0	6,293
9.2 to 9.3	5.0	420
9.3 to 9.4	5.0	490
9.4 to 9.5	10.0	1,258
9.5 to 9.6	10.0	980
9.6 to 9.7	10.0	1,258
9.8 to 10.0	25.0	6,293

The example, which has shown the simplest scenario the KBES can analyze, illustrates the improvement made by the KBES over previous methods in that (a) it uses a formal and computerized method, (b) it diagnoses specific failure modes instead of producing a general hazard level, (c) it uses logic to diagnose hazards instead of aggregated weights or ratings, (d) it can diagnose multiple hazards at the same site that the old manual method could not do, (e) it assigns an extent of failure and an explicit probability of failure that were absent from the previous method, (f) it explains why a particular hazard and risk was diagnosed, (g) it operationalizes the concept of vulnerability in terms of road standard and corresponding design, and (h) it performs hazard assessment in a

TABLE 5 EXPLANATION OF HAZARD DIAGNOSIS

Location: 7.0 to 7.3 km	Location: 7.3 to 7.7 km
Type of failure: Planar rock slide	Type of failure: Wedge failure
Extent of failure: Small	Extent of failure: Medium
Explanation	
The slope angle is between 36° and 45°	The slope angle is between 36° and 45°
The dip slope is the natural slope	The natural slope is not the dip slope
The alignment is parallel to the strike of beds	The alignment is perpendicular to the strike beds
More than three joints sets present	
The rock is fractured slate	More than three joint sets present
The main boundary thrust is nearby	The rock is fractured slate
A medium rock slide is at 7.2 to 7.3 km	The main boundary thrust is nearby
Vegetation is sparse brushes	Vegetation is sparse brushes
High rate of precipitation is the main trigger	High rate of precipitation is the main trigger
No soil cover	No soil cover
Seismically active region	Seismically active region

way that enables risk assessment. The KBES can also assess risk when hazard recurrences are of concern and when different countermeasures may be taken in the event of a hazard.

CONCLUSION

A KBES, which is currently at the stage of a developmental prototype, has been developed for hazard and risk assessment of low-volume hill roads in Nepal. This KBES, by formally organizing ill-structured hazard and risk assessment knowledge, provides a convenient way for nonspecialist practitioners to incorporate the important considerations into the practice of hill road construction in Nepal. Also, the KBES has contributed a standardized method of integrating hazard and risk assessments to the state-of-the-art of this field in Nepal.

Work is proceeding to expand the knowledge base and to adjust the control strategy accordingly. This work is oriented towards the production of an operational prototype and a commercial level system.

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Application of New Rock Drilling Technology to Forest Road Construction in Coastal British Columbia

D. M. BENNETT

In 1988, the Forest Engineering Research Institute of Canada initiated a series of studies to evaluate the effect of state-of-the-art drills on the techniques and costs of constructing low-volume forest roads in coastal British Columbia. The fact that rubber-tired units are more mobile than conventional pneumatic tank drills has changed many aspects of constructing coastal logging roads. Total ownership and operating costs for the rock drills studied varied from $127 to $182 per hour (in Canadian dollars). Average productivity varied from 97 to 128 m drilled per shift, and cost per meter drilled ranged from $9.15 to $14.43. Rock drill utilization levels ranged from 31 to 46 percent. The relatively low utilization levels result from lengthy nonmechanical delays associated with the overall forest road construction process; planning, work procedures, and the organization of crews and equipment can influence these figures.

The western Canadian province of British Columbia contains an extensive network of low-volume roads that have been constructed for timber extraction, and hundreds of millions of dollars are spent annually to construct new roads. Complete data are not available, but the Forest Engineering Research Institute of Canada (FERIC) estimates that in 1988 the province's forest companies, and to a lesser extent the provincial Forest Service, built approximately 2500 km of unpaved forest roads in the coastal region of British Columbia. In addition, several thousand kilometers of existing road are maintained on a regular or sporadic basis.

The most difficult conditions for forest road construction are found in the coastal region. Timber harvesting systems on the coast use off-highway logging equipment that requires forest roads to be constructed to a 165-tonne load rating. The terrain is mountainous, annual rainfall often exceeds 4500 mm, and rock excavation by drilling and blasting is continually required. As a result, average forest road construction costs range from $50 000 to $150 000 per kilometer (in Canadian dollars).

Because of escalating costs of drilling and blasting, there has been a resurgence in development of new rock drilling equipment for constructing low-volume forest roads in coastal British Columbia. Rubber-tired hydraulic rock drills were successfully introduced to the forest industry in 1984. New equipment configurations include pneumatic or hydraulic drifters mounted on tracked or rubber-tired carriers. However, little information is available to guide forest road builders in selecting the most cost-effective equipment complement for a particular set of road-building conditions. In order to assist the forest industry, FERIC initiated a series of case studies in 1988 to evaluate the performance and utilization of state-of-the-art drilling equipment in British Columbia and to provide quantitative information on related productivity and costs (*1–4*). Current road-building practices in British Columbia are reviewed and results of these case studies are described. Rock drill productivity, utilization levels, and mobility are related to the goal of achieving the best overall cost per meter drilled.

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CURRENT FOREST ROAD CONSTRUCTION PRACTICES IN COASTAL BRITISH COLUMBIA

Coastal British Columbia is dominated by glaciated valleys and steep, rugged mountains mantled with thin soils (<1 m deep) over hard and massive bedrock. On Vancouver Island, where the rock-drill studies took place (Figure 1), bedrock formations consist mainly of thick volcanic flows (usually basaltic), and intrusive igneous rocks that are typically granodiorites, with lesser amounts of sedimentary formations such as limestone. The adjacent mainland coast is dominated by massive intrusive igneous rocks, which are principally granodiorites.

Drilling and blasting is an integral part of coastal forest road construction, so a typical road construction team usually consists of a primary subgrade machine (usually a hydraulic excavator), a rock drill, and sometimes a crawler tractor. For many years, the excavator has been the primary subgrade machine used in coastal British Columbia. When used efficiently, the excavator produces high-quality subgrade and substantially reduces, or eliminates, the need for expensive ballasting (road surfacing) that may be required to meet load-carrying specifications. It performs several functions in the construction process, which can be described in three distinct phases. In the first phase, the excavator "pioneers" ahead on the right-of-way, establishing the road centerline, moving the felled and bucked timber to the side for later retrieval, and removing tree stumps. During the second phase, the excavator strips the overburden from the underlying mineral soil and deposits it well away from the road to ensure that the organic overburden is completely separated from the mineral soil. In the final phase, the machine excavates and shapes the road prism to the required specifications, creates a ditch, and installs drainage structures.

The crawler tractor has taken on a secondary role and is often not needed to produce a finished road. It can move

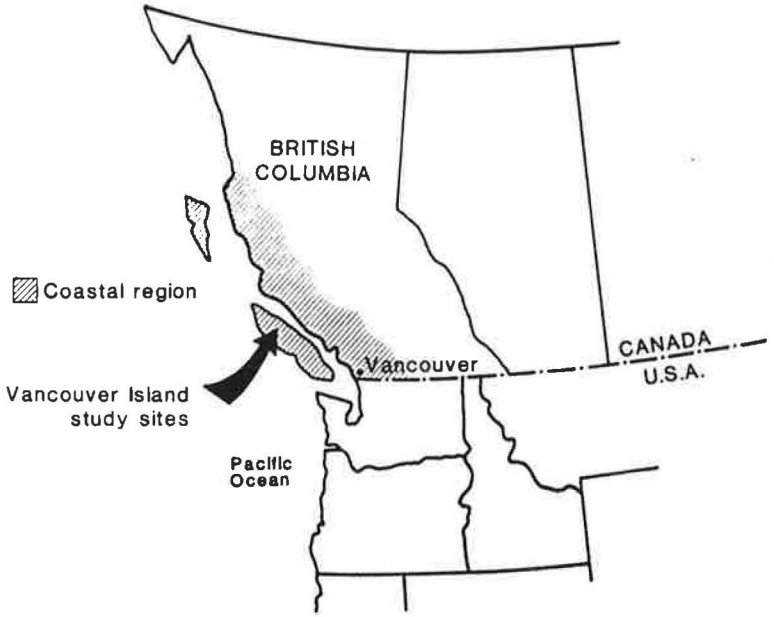

FIGURE 1 FERIC's rock drill study sites.

excess material from the cut to fill areas and grade the road surface, but often an excavator and rock drill can produce a finished road at a lower cost per kilometer without the crawler tractor.

When the excavator encounters an impassable section of rock in the road right-of-way, the rock drill moves to the site and drills enough rock to allow the excavator to resume work. In situations where the excavator bypasses rock outcrops and continues with the initial phase of subgrade construction, the rock drill works concurrently to widen the subgrade and drill rock for ditch excavation.

Blasting methods typically involve some form of controlled blasting in which individual boreholes or rows of holes are initiated in a sequence that controls rock movement and achieves a finer fragmentation. The purpose of controlled blasting is to minimize hazardous flyrock and reduce the waste of suitable construction material. The most widely accepted detonation system uses nonelectric (NONEL) blasting caps and detonating relays in conjunction with cast primers and detonating cord. An ammonium nitrate and fuel oil (ANFO) blasting agent is the most common explosive used because its cost is approximately 25 percent of the cost of packaged dynamites. The wet conditions on the West Coast often require the use of water-resistant ANFO products, borehole dewatering techniques, and borehole liners. ANFO prills can be poured into downholes or loaded into horizontal (lifter) holes with a pneumatic blasthole charger.

Road construction costs for most coastal forest companies decreased significantly between the late 1970s and the mid-1980s. This cost reduction was influenced by several factors, mainly:

- Increased use of ANFO rather than expensive packaged dynamite products;
- Improved overburden stripping techniques and greater use of local materials; and
- Reduced overhead costs necessitated by restraint measures during an economic recession in 1981–83.

CURRENT ROCK DRILLING EQUIPMENT

The pneumatic tank drill has long been the standard rock drill used in the forest industry, but now hydraulic tank drills, hydraulic crawler drills, and hydraulic rock drills mounted on rubber-tired carriers are gaining acceptance. The pneumatic tank drill has evolved through several models. More than 200 units are in use today, mainly in the Pacific Northwest. The most recent version is the Finning M32FA, which uses a Gardner-Denver PR66 valveless pneumatic drifter, a 380-L/sec (800-ft^3/min) compressor, and a tracked carrier built with military-type components.

Ten to 15 years ago, hydraulic tank drills were used (to a limited extent) for forest road construction, but they did not gain widespread acceptance. However, a manufacturer in British Columbia recently developed a new version that uses Tamrock hydraulic drilling components mounted on a hydrostatically driven tracked carrier. Four of these units are in use, all in British Columbia.

Hydraulic crawler drills are widely used in the construction and mining industries but have limited application in the forest industry because the carriers are not designed to negotiate the rugged subgrade conditions typical of logging roads. Some contractors prefer them because the capital cost is lower than the purpose-built rock drills. However, the crawler units may require modifications to meet local operational and safety standards.

Between 1969 and 1983, Atlas Copco Canada Inc. manufactured 36 pneumatic rock drills mounted on rubber-tired carriers. Some are still being used for building forest roads in coastal British Columbia and for secondary drilling in open pit mines in eastern Canada.

The rubber-tired hydraulic rock drill was developed with input from major British Columbia forest companies and has become increasingly popular in the last 5 years. Many major companies and some contract road construction firms have purchased these units. These machines represent state-of-the-art drilling technology. Three British Columbia manufacturers produce rubber-tired hydraulic rock drills, and 47 units are currently working in the province's forest industry. The carriers are similar to skidders used for timber harvesting in that they can easily negotiate rough subgrades and broken terrain. All manufacturers use maneuverable boom arrangements that enable the feed beam to be rotated 360 degrees in both the vertical and horizontal planes. The more maneuverable booms have increased the amount of downhole (vertical borehole) drilling in comparison to horizontal (lifter) hole drilling. This process in turn has introduced other efficiencies in blasting methods. It is generally agreed that downholes permit better control of blasts. Also, operations can use a higher percentage of ANFO because the blasting agent is easier to load into vertical boreholes.

The mobility of rubber-tired drills is considered important in forest road construction because they travel more quickly than tracked drills, thus facilitating movements between work sites. This can translate into more available drilling time. In some situations, one rubber-tired drill can do the work of two tank drills because the high travel speeds permit it to service neighboring work sites without delaying or idling other road-building equipment. The extra time and cost of moving tracked drills between work sites on lowbed truck trailers is also eliminated.

FERIC'S ROCK DRILL STUDIES

Machine and Site Descriptions

Four case studies have been completed on four state-of-the-art rock drills working in the logging operations of major forest companies on Vancouver Island (Figure 1). Most phases of road construction and logging in these operations used company-owned equipment and company-employed crews. All the operators were experienced and knowledgeable drillers and blasters.

Three hydraulic rock drills mounted on rubber-tired carriers were monitored: a Tamrock Logmatic at the Kelsey Bay Division of MacMillan Bloedel Limited (MB) (Figure 2); a Cypress Roc-Champ at MB's Eve River Division (Figure 3); and a Finning RTD528 Rock Master drill at the Englewood Division of Canadian Forest Products Limited (Figure 4). The fourth drill, a Finning M32FA pneumatic rock drill mounted on a conventional tank-type carrier, was monitored at MB's Cameron Division (Figure 5).

Study Method

The method developed by FERIC for studying the drills involved working closely with each operator for a period of 3 to 4 months. A DSR Servis Recorder mounted in the cab produced detailed charts of drilling activities and machine travel for each shift. The driller documented all nonmechanical delays and breakdowns, daily production, and an estimate of rock conditions for each shift worked. In order to collect more-detailed information about the drilling cycle and penetration rates, a detailed timing study using stopwatches was conducted on each machine for a period of 1 week.

FIGURE 2 Tamrock Logmatic at Kelsey Bay.

RESULTS AND DISCUSSION

Operating Conditions

In all case studies, the drills were used mainly for drilling grade rock (i.e., the rock within the road prism that must be excavated to produce the finished subgrade) during the course of normal subgrade road construction (Table 1). In the Kelsey Bay study, other occasional work duties included drilling quarries for road surfacing material, drilling ditch lines, and widening existing roads.

A three-person crew was employed at the Eve River study site to perform drilling and blasting activities, whereas two-person crews (driller-blaster and assistant) were used at the other study sites. The three-person crew consisted of a full-time drill operator and assistant as well as a full-time blaster. The blaster transported explosives, loaded explosives into boreholes, and fired the blasts. The goal was to increase drilling production by reducing the amount of nonmechanical delay time incurred by having the drill operator load explosives and fire blasts, as is the case with two-person crews.

The size of drill bits used varied (Table 1). In the Cameron and Eve River studies, 76-mm button bits were used exclu-

FIGURE 3 Cypress Roc-Champ at Eve River.

FIGURE 4 Finning RTD528 at Englewood.

sively, but a mix of 64- and 76-mm bits was used at the Kelsey Bay and Englewood sites.

Time Distribution

Figures 6–9 show time distributions for each drill studied. In the Kelsey Bay study, drilling accounted for 38 percent of total time, mechanical delays (repair time plus waiting time for mechanics and parts) made up 10 percent, and nonmechanical delays accounted for 52 percent of the total time studied. For the Cameron case study, 31 percent of the total time was spent drilling, 18 percent was for mechanical delays, and 51 percent was for nonmechanical delays. The breakdown for the Eve River study was 46 percent for drilling, 19 percent for mechanical delays, and 35 percent for nonmechanical delays. The experience during the Englewood study was 32 percent of total time for drilling, 10 percent for mechanical delays, and 58 percent for nonmechanical delay time. (During the latter half of 1989, when the Englewood case study was conducted, road construction conditions were good. The drill was idle for a number of shifts because no rock was available for drilling. If these atypical shifts are not included in the utilization calculations, drilling time would be approximately 40 percent, mechanical delays 13 percent, and nonmechanical delays 47 percent of the total time studied.)

The high proportion of nonmechanical delay time is characteristic of rock drill use in logging road construction. Loading boreholes and blasting was the largest single reason for drilling delay in all the studies, accounting for 26 percent of total shift hours at Kelsey Bay, 24 percent at Cameron, 10 percent at Eve River, and 22 percent at Englewood. The use of the three-person crew at Eve River is reflected in the relatively low proportion of loading and blasting delay time recorded at this site.

FIGURE 5 Finning M32FA at Cameron.

Waiting for the subgrade machine to prepare more rock for drilling was the second largest cause of nonmechanical delay time in the Kelsey Bay, Cameron, and Eve River studies. In the case of the Englewood study, the second largest cause was the machine being idle because of a lack of rock (16 percent). The Other category shown in Figures 6–9 includes delays such as machine travel between road headings (work sites), drill operators' blasting dangerous trees (e.g., snags) during the timber-felling phase of the logging operation, marking of borehole locations on the ground, and waiting for more right-of-way timber to be felled.

Utilization and availability are presented for each machine in Table 2. Rock drill utilization, which is the time spent on the drilling function, includes drifter percussion time and the time spent positioning for each borehole. Machine availability excludes waiting time for parts and mechanics, as well as repair time.

The drilling time, or production machine hours (PMH), identified on the Service Recorder charts also included minor delays to position the boom and the machine for drilling the next borehole. During the detailed-timing period, this drilling time was further subdivided into actual drifter percussion time and positioning time, which are the two main elements, and other miscellaneous activities. The results of the detailed timing period are presented in Table 3 for the four study sites. The ratios of percussion time to positioning time, when applied to the shift-level data, suggest that the drifter operated for about 24 percent of total shift hours in the Kelsey Bay study, 20 percent in the Cameron study, 34 percent in the Eve River study, and 18 percent in the Englewood study.

Machine Travel Between Road Headings

When evaluating rock drill utilization, considering the number of road headings under concurrent construction is important. In favorable situations, where distances between road headings are short, a highly mobile rock drill may be able to service two or more road headings efficiently without idling the

TABLE 1 CONDITIONS DURING STUDIES

	Study sites			
	Kelsey Bay	Cameron	Eve River	Englewood
No. of operating shifts	60	68	47	71
Total shift hours (TSH)	467.9	468.2	354.5	556.3
Bit type [a]				
64-mm button	73%	0%	0%	45%
76-mm button	27%	100%	100%	55%
Activity [a]				
Grade rock	87%	100%	100%	100%
Quarry	3%	0%	0%	0%
Road maintenance	10%	0%	0%	0%

[a] Expressed as a percentage of total metres drilled.

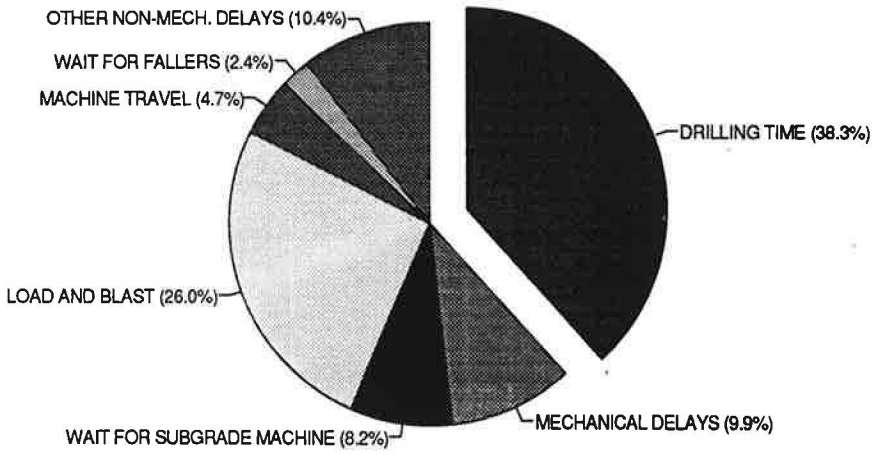

FIGURE 6 Time distribution for the Kelsey Bay study.

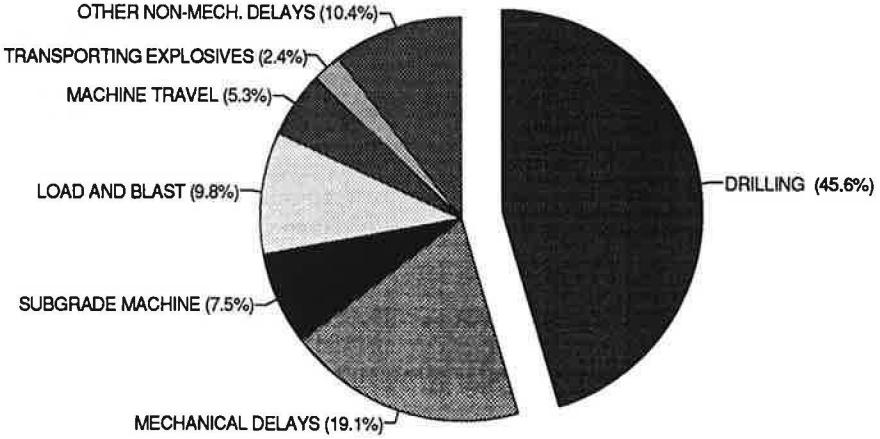

FIGURE 7 Time distribution for the Eve River study.

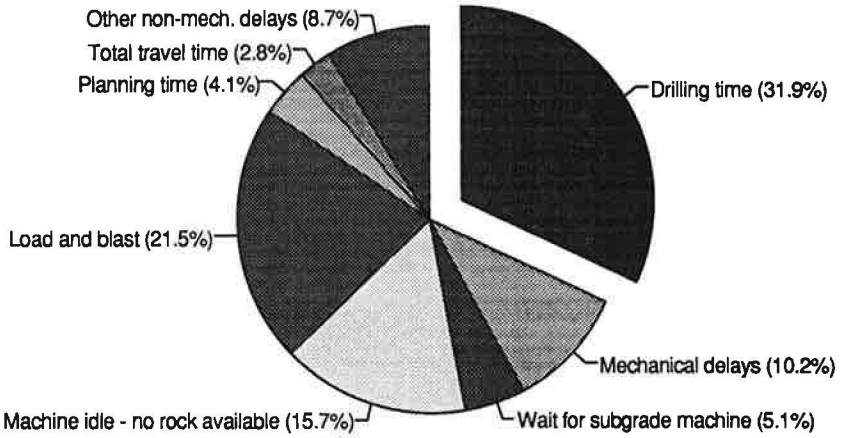

FIGURE 8 Time distribution for the Englewood study.

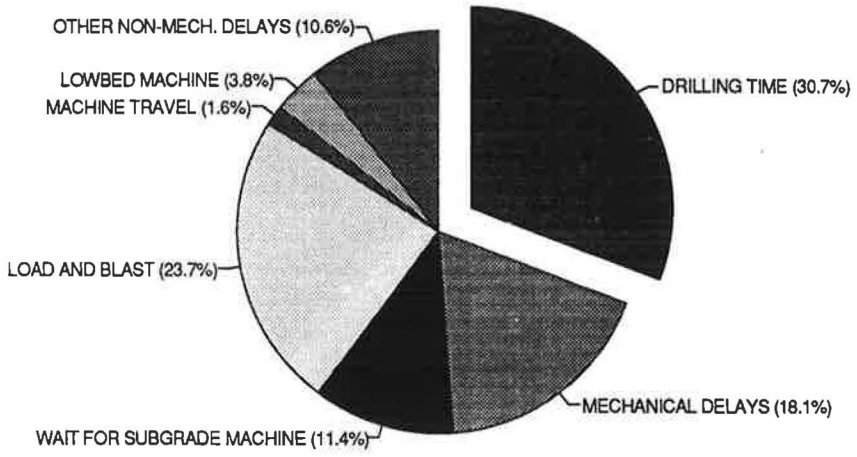

FIGURE 9 Time distribution for the Cameron study.

subgrade machines. However, the amount of time the drill spends traveling must be weighed against lost productivity (drilling time) when determining the most efficient use of equipment and labour.

Machine travel time (Figures 6–9) accounted for 5 percent of total shift hours for the Kelsey Bay, Cameron, and Eve River studies, and 3 percent of total shift hours for the Englewood study. These hours include travel time between road headings plus short moves within the work area. The Cameron results include the time that the tracked rock drill spent waiting for, and being transported on, a lowbed truck trailer. The rubber-tired units, however, traveled between headings under their own power.

Table 4 presents machine moves between road headings for each case study. The machine-move ratio (the number of moves per 100 shifts worked) indicates that machine mobility was an important element in the road-building strategies of the Kelsey Bay and Eve River operations. In both cases, the drills were utilized effectively without incurring excessive delay time for machine travel.

Productivity and Costs

For all the studies, 3-m downholes were the most popular method of drilling. The four measures of productivity presented in Table 5 are averages for the entire study period. Boreholes per shift and production per shift are conventional units used by the forest industry. Average number of meters drilled per shift ranged from 97.2 for the Cameron study to 128.0 at the Englewood location.

On a daily basis, production can fluctuate over a wide range. In the Englewood study, for example, the crew often drilled steadily for a few days before stopping the drill for a full shift, or more, to load and blast boreholes. As a result, there were 10 shifts in which more than 100 boreholes and more than 300 m were drilled.

Boreholes per PMH and production per PMH indicate the amount of drilling done when the machine was involved only in the drilling function. These figures exclude nonmechanical and mechanical delay time. Boreholes per shift and production per shift are strongly affected by machine utilization rates, whereas boreholes per PMH and production per PMH reflect factors such as the type and condition of the rock, the size of the drill bit used, the length and orientation of the boreholes, and the type and condition of the drifter itself.

Production per PMH ranges from 31.6 m per PMH at Eve River to 51.3 m per PMH at Englewood. The following factors probably contributed to this range:

- The Cypress Roc-Champ drill at Eve River encountered harder rock than the drills at the other study sites, and it drilled several long horizontal boreholes, which take more time to drill.
- The Finning Rock Master RTD528 drill at Englewod drilled short downholes exclusively.
- A larger 76-mm bit was used throughout the entire study period at Eve River, whereas a 64-mm bit was used during much of the Englewood study.

TABLE 2 ROCK DRILL UTILIZATION AND MACHINE AVAILABILITY

	Study sites			
	Kelsey Bay (%)	Cameron (%)	Eve River (%)	Englewood (%)
Rock-drill utilization	38	31	46	32
Machine availability	90	82	81	90

TABLE 3 ELEMENTS OF PRODUCTIVE MACHINE HOURS[a]

	Study sites			
	Kelsey Bay (%)	Cameron (%)	Eve River (%)	Englewood (%)
Percussion time[b]	63	66	76	58
Position for drilling	31	26	21	25
Other activities (e.g. (drill-rod changes, etc.)	6	8	3	17

[a] Productive machine hours (PMH) = Drilling time.

[b] As a percentage of productive machine hours (PMH).

TABLE 4 SUMMARY OF MACHINE MOVES BETWEEN ROAD HEADINGS

	Study sites			
	Kelsey Bay	Cameron	Eve River	Englewood
Total number of moves	16	6	12	5
Number of different road headings worked	8	3	8	4
Average distance travelled per move (kilometres)	8.8	21.5	19.6	15.6
Average time per move (hours)	1.0	2.1	1.2	1.2
Machine-move ratio $\frac{\text{(Total no. moves} \cdot 100)}{\text{Total shifts worked}}$	26.7	8.8	25.5	7.5

The pneumatic rock drill, at the Cameron study site, compared favorably to the rubber-tired hydraulic machines in terms of production per PMH and boreholes per PMH.

Hourly machine costs (Table 6) and unit drilling costs (Table 5) were calculated for all case studies using one standard method and may not represent the actual costs experienced by the users. All costs are based on figures available as of July 1990, and they include the labor rates in effect at that time. Additional information on costing is presented in the individual case study reports.

Two basic unit costs for each case-study period are presented in Table 5: Unit cost (Canadian dollars per borehole) and Unit cost (Canadian dollars per meter/drilled). Machine cost per hour was multiplied by total shift hours and then divided by the total number of boreholes or meters drilled to yield these costs. In the case of the Eve River study, the extra cost of the full-time blaster was also added to the hourly machine cost. For the Cameron study, the extra cost of transporting the tracked drill with a lowbed truck trailer was included in the unit cost calculations. The lowest costs per borehole were recorded by the Kelsey Bay and Cameron case studies. The average borehole length was greater at the Cameron study site, resulting in the lowest overall cost per meter drilled ($9.15 per meter). The highest cost per meter drilled was recorded in the Eve River study ($14.43 per meter). However, the Roc-Champ drill performed most of the rock drilling duties in an operation that used three excavators to build 26 km of road during the year in which the study was conducted. The three-person crew used at Eve River contributed to higher rock drill utilization, as shown in the time distribution results, but it also added to the drilling costs.

TABLE 5 SUMMARY OF ROCK DRILL PRODUCTIVITY AND COSTS FOR SHIFT-LEVEL STUDIES

	Study sites			
	Kelsey Bay	Cameron	Eve River	Englewood
No. of boreholes drilled	3 025	2 163	2 077	3 368
Boreholes per shift	50.4	31.8	44.2	47.4
Boreholes per PMH [a]	16.9	15.1	12.9	19.0
Unit cost (C$ per borehole)	26.43	27.95	35.47	30.07
Production (metres)	7 582	6 607	5 105	9 091
Production per shift (metres)	126.4	97.2	108.6	128.0
Production per PMH (metres)	42.3	46.0	31.6	51.3
Unit cost (C$ per metre drilled)	10.54	9.15	14.43	11.14

[a] Boreholes/PMH (productive machine hours) = Average number of boreholes drilled for every hour the machine was actually employed on the drilling function.

Other Observations

The rubber-tired hydraulic rock drills are more technologically and ergonomically advanced than the traditional pneumatic tank drill. Cleaner, quieter, dust-free cabs; air conditioning; better control systems; and improved visibility make these machines more popular with operators. The more maneuverable booms and boom heads improve the operator's ability to position the feed beam for drilling. Of the operators who have worked steadily on a rubber-tired hydraulic machine, few state a preference for going back to a pneumatic tank drill.

The importance of the operator to rock drill utilization and performance cannot be overstressed. Proper training of operators and mechanics is especially important with the more complicated rubber-tired hydraulic drills. The organization and coordination of other road crews and equipment, the number of road headings available, and the amount of rock encountered on those headings are also important to rock drill productivity and should be considered when evaluating machine performance.

The superior mechanical efficiency of hydraulic drifters results in faster penetration rates than could be achieved by the pneumatic drills that were common to the industry 15 to 20 years ago. However, the difference is not as great when compared to advanced pneumatic drifters coupled with compressors in the 380-L/sec class. Hydraulic drifters are powerful and relatively quiet, but state-of-the-art pneumatics can produce at an acceptable rate in the forest road construction application. Detailed timing results showed that rock conditions alone can have a greater effect on penetration rates than the type of drifter.

At Eve River, a three-person crew was employed in an effort to increase utilization. This study recorded the highest utilization level but not the lowest overall cost per meter drilled. The reasons for the latter include ownership and operating costs of the particular machine, difficult rock conditions, and the extra cost of employing the full-time blaster. It would appear that the anticipated benefits of a larger crew may often be less than the extra costs incurred. If extra care is taken to assign a knowledgeable and qualified assistant to a driller/blaster, then significant gains in utilization may be possible

TABLE 6 MACHINE COSTS

	Study sites			
	Kelsey Bay	Cameron	Eve River	Englewood
Machine				
Model	Tamrock Logmatic	Finning M32FA	Cypress Roc-Champ	Finning RTD528
Carrier	Rubber-tired	Tank-type	Rubber-tired	Rubber-tired
Drifter	Hydraulic	Pneumatic	Hydraulic	Hydraulic
	Tamrock HLR 438L	Gardner-Denver PR66	Atlas Copco 1238 ME	Gardner-Denver HPR1HA
Purchase price [a]				
(C$)	350 000	278 000	394 000	403 000
Hourly ownership and				
operating cost (C$)	170.87	126.52	180.61	182.05

[a] Price as of July 1990.

without resorting to a three-person crew. A well-trained assistant can reduce nonmechanical delay time by occasionally operating the drill and relieving the regular operator, or by loading explosives into previously drilled boreholes while the driller continues to operate the machine.

Thorough planning of road construction activities and coordinating them with other phases of the logging operation is essential for reducing nonmechanical delay time. Drilling operations that are forced to restrict blasting times because of the proximity of logging crews and equipment have lower productivity. It is also important to ensure that enough right-of-way timber is felled in advance of construction.

The results of these case studies suggest that the conventional pneumatic tank drill yields the lowest overall cost per meter of borehole. However, the primary objective in forest road construction is to achieve the lowest possible cost per kilometer for an entire road-building operation. This is a vital point. A high cost per meter of borehole does not necessarily produce a high cost per kilometer of finished road. However, an analysis of productivity and unit costs is needed to compare rock drills and to determine the correct rock drill complement for a particular set of operating conditions.

For a logging operation with one road-building team and requiring one rock drill, a state-of-the-art pneumatic tank drill can drill effectively and give a lower cost per meter than rubber-tired hydraulic machines. It is more difficult to estimate the long-term drilling requirements in a larger operation with multiple work sites. Long-range road-building plans must be studied to determine whether one rubber-tired unit can replace two or three tracked machines on a consistent basis. If multiple nearby road headings can be continually serviced by one rubber-tired drill, then obviously the cost per meter drilled will be lower than if two tank drills are used. A combination of one, or perhaps two, rubber-tired hydraulic drills and one or more tank drills is possibly the best drill complement in a large operation. The rubber-tired hydraulic drills could work in areas of multiple road headings containing isolated or discontinuous rock, and the pneumatic tank drills could work in areas of heavy rock requiring continuous drill-and-blast sequences. Other alternatives, such as the new hydraulic tank drills and hydraulic crawler rigs, may be cost-effective as well.

CONCLUSIONS

Rock drilling technology for forest road construction is constantly evolving and it appears that no one type of rock drill is the most cost-effective in all situations. The utilization, productivity, and cost information determined in FERIC's studies will help forest road builders choose the correct rock drilling arrangement for a particular application.

The studies indicate that the most significant gains in rock drill productivity and costs can be made by improving rock drill utilization, that is, increasing the amount of drilling time relative to idle time. When actual percussion time for drifters is in the range of 18 to 34 percent of total shift time, the effects of factors such as penetration rate and rock type are not as significant as the gains possible from small increases in utilization. In FERIC's studies, rock drill utilization ranged from 31 to 46 percent. It is not easy for forest road builders to make significant improvements in rock drill utilization because other phases of the logging operation strongly influence road-building operations. However, a goal of 50 percent for rock drill utilization would be a reasonable target for operators and supervisors to work toward.

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Locating Subsurface Gravel with Thermal Imagery: A Progress Report

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A method is developed for locating gravel deposits in vegetated areas using six-band thermal imagery. The geologic history of the region is reviewed to select a potential area of study. An overflight is made using a six-band multispectral scanner. The data are displayed on a monitor and a spectral signature is selected from areas of known gravel deposits. A typical gravel signature demonstrates a strong absorption feature at the 9.2-μm wavelength. The data are then processed with a computerized system and redisplayed to delineate additional areas in the scene with a similar spectral signature. The display is transferred to a map, and the signature areas are located in the field. Exploratory drilling is used to verify the presence of gravel and to determine the thickness and extent of the deposit. During 1989 and 1990, gravel deposits totaling 1 million m^3 were discovered on national forest land near Alexandria, Louisiana, using this method. Concurrent studies are seeking to understand how the thermodynamic responses of materials contribute to the overall spectral radiant emittance in the thermal infrared spectrum. Ground instrumentation has been installed to support this effort. More time and effort are needed to advance the understanding of the physics of the method. The techniques developed can serve as a basis for further studies, directed towards extending the technology to a broad range of environments.

The need for gravel is as well established as the general knowledge of its abundance in certain areas across the Gulf of Mexico coastal plain, including lands within several national forests. Exploratory drilling between 1975 and 1988 had revealed few exploitable deposits. During this 14-year period, only five major deposits, totaling less than 1 million m^3 of gravel, were located on national forest land. Efforts to locate gravel deposits using thermal imagery began in 1983 with the acquisition of 30-m resolution thermal infrared multispectral scanner (TIMS) imagery over the Kisatchie National Forest near Alexandria, Louisiana. A report on preliminary results was published in 1986 (*1*). Shortly after the preparation of that report, data processing was halted because of hardware problems. In May 1987, processing of the original imagery was resumed. In October 1987, the Forest Service Southern Region (R8) and NASA Stennis Space Center (SSC) Science and Technology Laboratory (STL) prepared a joint proposal to further develop TIMS aggregate exploration technology for submission to the NASA Earth Observations Commercial Applications Program (EOCAP). Other early efforts were centered on organizing and coordinating software capabilities of STL and R8. Two scenes of the original TIMS imagery from the Kisatchie National Forest, 15 km square, were rectified to a standard map base to evaluate new aircraft image-to-ground coregistration software at NASA/STL. Subsequently, additional scenes were rectified at the Forest Service facility in Atlanta, Georgia. Rectifying the imagery to map coordinates greatly facilitated location of image points in the field for verification of results. It also allows entry of potential gravel sites into a geographic system with map data. The development of image rectification and improvements in image processing techniques, have resulted in the discovery of 1 million m^3 of gravel deposits on the Kisatchie National Forest during 1989 and 1990.

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ENVIRONMENTAL SETTING

Some understanding of the origin of emplacement of these gravel deposits provides a model for understanding their occurrence and improves the chances for the success of gravel exploration efforts.

Deposition of these Pleistocene gravels occurred after periods of glacial buildup when ocean levels were down and the main river channels had cut deep gorges, leaving the subsidiary streams with increased gradients to reach the main channels. Higher velocities in these steeper reaches increased the bed loads and separated fines from gravels. Wherever the gradient flattened, coarse material settled out and formed banks of gravel as the fines washed downstream. During the warm interglacial periods that followed each glaciation, melting ice brought heavy rainfall and torrents of runoff carrying huge sediment loads that separated into gravel banks below these steeper reaches where abrading streams developed. As the oceans rose again, filling in the main channels, these abrading areas were gradually flattened and covered over by progressively finer material. Isostatic uplift caused by the receding glaciers, and the subsequent erosional processes, often left these erosion-resistant gravels on the ridge tops of present-day topography, hidden by thin layers of sand and vegetation. Widespread scattered surficial gravel 1 or 2 in. in thickness overlying deep sand complicates the process of discovery.

This description represents the environmental setting in which, because of their ready availability and effectiveness in performing reconnaissance surveys of large areas, various electrooptical remote sensing methods are currently being tested. Of these methods, thermal infrared remote sensing techniques have shown the most promise. The use of NASA's TIMS has produced imagery that exhibits a promising correlation with known subsurface gravel deposits.

IMAGE ACQUISITION

The TIMS was developed by STL as a definition tool for future satellite-borne, geology-oriented sensors. First flights were made in summer of 1982, and first imagery for the U.S. Department of Agriculture, Forest Service (USFS) was obtained in October of 1983 over the Kisatchie National Forest in Louisiana, and 1 year later over the Desoto National Forest in Mississippi and the Okmulgee district in Alabama (*1*). More recently, in September 1989, additional imagery was obtained over the Desoto National Forest. TIMS is an electrooptical line scanner currently operating in the airborne mode, sensing in six narrow bands in the range of 8.2 to 12.2 μm.

Band	*Spectral Coverage, μm*
1	8.2 to 8.6
2	8.6 to 9.0
3	9.0 to 9.4
4	9.4 to 10.2
5	10.2 to 11.2
6	11.2 to 12.2

Average spectral sensitivity is approximately 0.1°C. The instrument was recently completely dismantled and refurbished by STL to ensure its continuing standard of performance. The sensor is flown in a Lear 23, a C130, or a modified U2 aircraft. Instantaneous field of view (IFOV) and lateral coverage of flightlines depend on aircraft altitude. The altitude of these aircraft may be varied within operational limits, between 2000 and 24 000 m, to provide an IFOV ranging from 5 to 60 m. The initial imagery obtained for the USFS in 1983 and 1984 was from the Lear 23 at an altitude of 12 000 m above terrain and provided data with 30-m pixels and a swath width of approximately 18.7 km (*2*).

The six bands of data obtained from the TIMS operation are in digital format. This format provides a relative measure of the emissivity from the ground surface soil minerals at each of the six wave lengths within the midinfrared range, and makes it possible to plot a spectral signature for each 30-m pixel. A 30-m pixel provides ample detail of surface deposits, which often spread over broad areas with little variation.

Good imagery for gravel exploration has been obtained in the predawn hours of late September to early October, at altitudes of 9000 to 12 000 m, following periods of at least 10 days without rainfall.

The season, time of day, weather, and altitude have all been found to be important factors in obtaining imagery that can be successfully interpreted. The season affects the relative solar heat storage and thermal emission from surface deposits. The time of day affects the direction of heat flow in the earth. Dry weather during the period immediately before image acquisition is essential to the differentiation of materials. These items will be discussed in greater detail later.

IMAGE PROCESSING

Since preparation of the first report (*1*), the hardware used for processing the imagery has been upgraded from a Data General S250 to an MV15000 with a 600-megabyte fixed disk and two 200-megabyte removable disk drives. A second 600-megabyte fixed disk was installed early in 1989. This increased data processing capacity has permitted a study of available data on a scale that was previously not possible. Because of delays in completion of software and hardware installation and procedure development, processing of NASA tapes obtained in 1983 and 1984 containing TIMS early-morning 30-m resolution imagery from Louisiana, Mississippi, and Alabama did not get underway until May 1987. Installation of Erdas software, Version 7.2, during summer 1987 greatly improved image processing capabilities.

The SIXBP program (six-band plot), which provides a display on the monitor screen of the six band values of the pixel at the cursor location together with a plot of the signature placed on the RGB screen, was developed early in 1989 using the ERDAS Toolkit (Figure 1).

Quartz gravel and coarse sand provide a unique spectral signature in the midinfrared range spanned by the TIMS, with a strong absorption feature in Band 3 (*3*). Initial effort at interpreting the imagery consisted of displaying Bands 1, 3, and 6 on the RGB monitor where known gravel pits were seen to be brightly delineated; known gravel deposits showed a stronger absorption in Band 3 than adjacent soil deposits. An image processing technique was then developed to attempt to identify all areas containing gravel deposits. The technique involves isolating a slice of data that contains a desired spectral signature, producing an image that includes only those areas exhibiting the gravel signature, and displaying the potential gravel sites with background reference data to identify areas for field evaluation.

The visual reference to the signature form provided by the SIXBP program was extremely helpful in making quick evaluations of potential gravel sites, and greatly speeded up the study process. By the summer of 1989, an improved six-band gravel signature for lightly timbered areas that produced spectacular results in the field was identified (Figure 2). This signature aided in the discovery of 600,000 m^3 of gravel deposits during 1989 and 1990. In 1990, a second signature was identified (Figure 2) for moderately timbered areas. This signature has been used to locate an additional 400,000 m^3 to date. Many areas identified by the imagery as potential gravel deposits have yet to be explored.

A major goal of the investigation is to develop gravel deposit inventory maps for each national forest. In December 1989, an ERDAS image file of a portion of the Kisatchie National Forest previously rectified by STL/NASA, and processed using the revised procedures, was converted to an ARC–INFO polygon file. A map was subsequently produced on the Calcomp 5835XP high-speed color electrostatic plotter on which the potential gravel deposits were plotted over cartographic data derived from primary base maps for the forest (Figure 3). The superior quality of this product ensures its future use as both an office reference and a tool for field exploration.

FIELD RECONNAISSANCE AND EXPLORATORY DRILLING

Once a potential gravel signature is identified and a map of signature areas developed, the next step is to locate those areas in the field. Remote sites should be plotted on a U.S. Geological Survey quad to take advantage of topographic data. Geographic details such as roads, prominent topography, and drainages offer the best guides to field location.

FIGURE 1 A TIMS thermal image of the Williana, La., area with a display of the spectral signature for the pixel under the cursor (white cross), produced with the SIXBP software program.

Where these are lacking, accurate location is difficult or impossible except through the use of Global Positioning Satellite (GPS) systems and accurately georeferenced data.

In unroaded areas, when the general vicinity is known the area must be traversed on foot to look for signs of surface gravel. A thorough reconnaissance can involve several hours of hiking. Because pine straw and dry leaves will obscure surface minerals, the ground cover must be kicked aside at intervals. Often even surface gravel will be covered by several inches of sandy soil, and the only clues are offered by animal burrows, where a few pieces of gravel can usually be observed adjacent to the burrow entrance if there is any gravel in the area. Plowed fire breaks, erosion troughs, and uprooted trees offer other opportunities to inspect subsurface material. Most subsurface gravel deposits will outcrop somewhere because of undulations in topography. If sufficient signs are present, a small bulldozer can be used to clear away enough vegetation to allow a drill rig to pass.

The surface area of a deposit is an important factor in determining quantities, and can be estimated by counting the signature pixels. A 1-m-thick deposit over 1 hectare contains 10 000 m^3 of sand and gravel. Initial drilling at intervals of 70 m to a depth of 5 to 8 m will establish approximate volumes.

A truck-mounted drill rig employing 6-in-diameter augers is used for the field exploration by the Forest Service in the Southern Region. This drill rig is located on the Kisatchie National Forest, and is operated by the work supervisor at the Catahoula District Work Center. A small bulldozer, normally used in site preparation, is available for clearing vegetation. For the past 10 years, 2 to 3 weeks each year have normally been scheduled for gravel deposit exploration. Since 1987, the drilling effort has been guided largely by TIMS imagery.

During the several weeks of drilling and reconnaissance in 1987 and 1988, only one sizable deposit of 50 000 m^3 was discovered. This deposit was surficial in nature, spread over 6 hectares of ground, with exposed banks of gravel on the steeper slopes. Other areas investigated during this period had only a few centimeters of gravel or less than 1 m of sand containing 5 to 15 percent gravel.

In 1989, following the improvements in processing techniques, six deposits of 7000 to 15 000 m^3 were discovered in

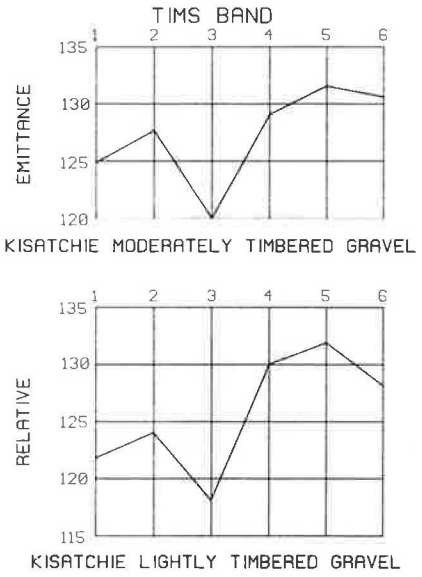

FIGURE 2 TIMS six-band spectral signatures for gravel deposits on the Kisatchie National Forest near Alexandria, La.

the Williana area during a 1-week period in July; and four deposits totaling an estimated 250 000 m^3 were discovered in the Pollock, Rock Hill, and Lake Kincaid areas during 1 week in December. During 2 weeks in August 1990, four deposits totaling an estimated 750 000 m^3 were discovered in the Pollock and Fishville areas. Thicknesses of these deposits varied from 1 to 6 m. All had less than 1 m of overburden. Two potentially large deposits, currently inaccessible to drilling except on the periphery, were identified in the Bentley area. A number of other signature areas remain to be investigated.

This wealth of discoveries occurring during 4 weeks of effort is entirely without precedent, and can only be attributed to the success of the TIMS spectral signature as a means of identifying gravel deposits.

THE SPECTRAL LINK TO MINERAL DEPOSITS

Three factors provide the predominate causes in the striking differences between spectral signatures of gravel deposits and deposits of other, finer-grained materials. These factors are the energy absorption of the quartz molecule in TIMS Band 3, the fraction of silt and clay in the material, and the thermal inertia of the material.

The energy absorption is caused by the stretching of the molecular bonds between the oxygen and silicon atoms that occurs in making up the SiO_2 molecule and its linkages. In order to maintain this configuration, the molecule must absorb energy from outside itself in the wave lengths associated with the TIMS Band 3. This process provides the striking signature associated with quartz. Clean, dry, coarse grains provide the strongest signatures. Impurities from clay minerals or other rock minerals, organic materials, excessively fine material, and moisture all tend to dilute the effect.

The association of coarse sand with gravel deposits is directly related to the velocity of flow in the channel. The depositional velocity for 2-cm-diameter particles has been found to be approximately 2.5 m/sec, and for 2-mm diameter, 0.5 m/sec. Particles finer than 2 mm resist settlement until still water is reached; thus there is little opportunity for the fine and coarse materials to intermingle (*4*). Coarse sand and gravel settle out in moving water, whereas fine sand, silt, and clay require a pond-like environment. This separation is further accentuated by shifts in channel location on the valley floor. A river carrying a coarse-grained load will develop a straight, shallow channel, but will change to a meandering, deep channel when the bedload becomes silt and clay (*5*). Thick gravel deposits are built up by fast shallow flows in wide, thin layers interspersed with coarse sand as the velocity varies with the seasonal runoff. When the upstream channel banks begin to provide finer-grained material, the river meanders and deepens, moving to an adjacent location and leaving the coarse-grained deposit intact.

The predominance of coarse sands found associated with gravel deposits identified by the TIMS gravel signatures indicates that these signatures are characteristic of coarse-grained quartz deposits, and conversely, that the strong energy absorption in TIMS Band 3 is maximized by coarse-grained quartz. These phenomena may be related to the total particle surface area in uniformly sized materials, which increases rapidly as the particle diameter drops below 2 mm. For example, the surface area increases from 10^3 to 10^6 cm^2 per cm^3 when the diameter drops from 3 to 1 mm. Photon energy is constant for each wavelength. The greater surface area scatters photon

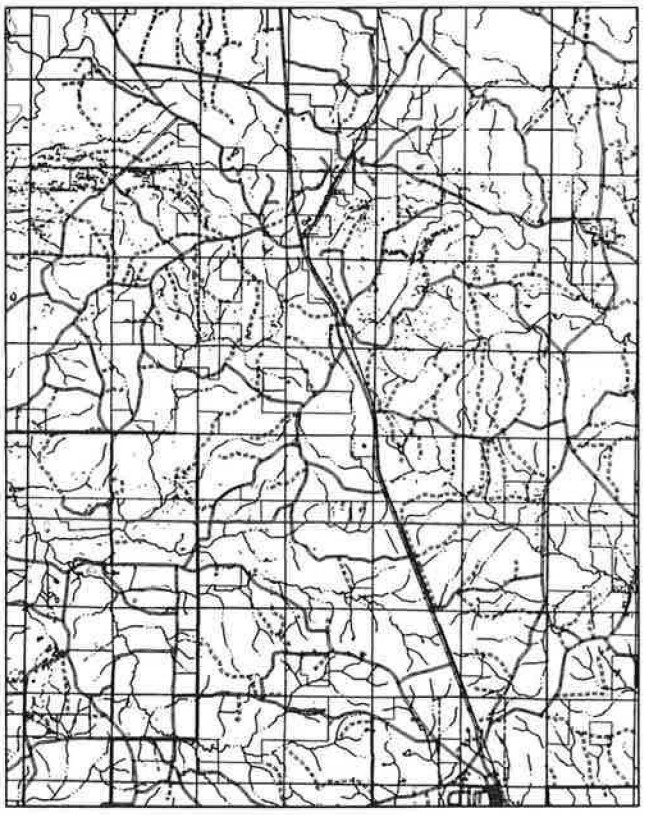

FIGURE 3 A gravel inventory map of the Williana, La., area. Hatched areas, indicating gravel deposits, were generated by processing rectified TIMS imagery and converting to ARC-INFO for plotting with an electrostatic plotter.

emission, causing a loss of energy and a corresponding shift to lower wavelengths. The greater surface area increases the energy available for absorption at the wavelength in TIMS Band 3. In the example, there are 1000 times more opportunities for a photon collision in the 1-mm particles than in the 3-mm particles. Silt particles have diameters in the same range as the radiation wave length, and scattering becomes the dominating effect (*6*). Thus a marked decrease in Band 3 emission (and correspondingly greater dip in the signature at this point) can be expected for the coarser-sized quartz materials, those whose diameters exceed 2 mm, as compared to the finer-grained quartz.

Thermal Inertia Factor

The thermal inertia of materials provides for striking contrasts in surface temperatures. Thermal inertia expresses the resistance of a material to temperature change. Materials of high thermal inertia change temperature only slowly, lagging behind changes in adjacent materials with low thermal inertia. A deposit of sand and gravel, for example, has a higher thermal inertia than a deposit of sand. Soil moisture, which increases as soil grain size decreases, also produces a higher thermal inertia.

Thermal inertia affects the temperature level of natural deposits. Periodic heating of the earth's surface from the sun provides two waves, the diurnal wave, affecting up to 1 m of depth, and the annual wave, affecting up to 20 m of depth. The actual depth of penetration depends on the properties of surface materials; denser materials generally have higher diffusivity and exhibit greater penetration. The diurnal wave is superimposed on the upper end of the annual wave, providing distinctly different surface temperatures for different materials, particularly when comparing day to night together with summer to winter.

The TIMS imagery obtained in 1983 to 1984 was flown in October at 2 a.m. on the assumption that the maximum annual ground temperatures had been attained at this time of the year, and during the early morning hours the maximum outflow of diurnal heat was occurring. The combination of maximum summer heating together with early morning cooling provides a unique effect associated with materials of high thermal inertia. Gravel and sand deposits always show cooler in the imagery than adjacent nongravel and sand deposits, although warmer than the damp bottom land. This perception was recently borne out by data from micrometeorological monitoring stations installed by NASA/STL. The resulting steep temperature gradient between the warm gravel body and its cool surface increases the rate of photon emission and further accentuates the Band 3 absorption dip.

Vegetation Factor

The spectral signature used to identify gravel deposits in this study is a mineral signature, related to vegetation only by the degree of dilution resulting from vegetation cover. The cover encountered in the 1983 imagery is by no means opaque. In the thickest forested areas, sunlight filters through to spot the shadows on the forest floor. The ground cover is made up of pine straw and dried leaves, neither of which can offer substantial obstruction to photon emission. Organic top soil is generally thin, especially on coarse-grained gravel deposits, and contains a large fraction of the parent soil exposed at the surface. Thus, photon emission from mineral grains can reach the scanner in flight overhead.

The 1983 imagery was acquired in predawn hours from an altitude of 12 000 m (with 30-m pixels). Vegetation has not been a problem with this imagery except in heavily timbered areas. In 1989, NASA acquired predawn imagery over the Desoto National Forest in Mississippi, at 9000 m (with 22.5-m pixels) and at 4000 m (with 10-m pixels), over the same terrain. Although the 9000-m imagery clearly identified several known gravel deposits both in timbered and open areas, the 4000-m imagery showed only vegetation. Thus, the threshold altitude for sampling mineral matter over vegetated terrain lies between 4000 and 9000 m.

This effect may be explained by the way the scanner records the predominant radiation spectrum. The primary sustained radiation source is the earth's mineral surface with its store of solar heat in the top few meters, represented in size to the scanner by the pixel size. Secondary radiation comes from cooler vegetation ranging in size from leaf to tree, often existing in clumps of trees interspersed with openings. When flying at lower altitudes, individual trees or clumps of trees are pixel-sized, and the earth contribution is limited by its small area; the flat vegetation spectrum predominates and the mineral signature is lost. At higher altitudes, individual trees and clumps are much smaller than the pixel; the earth openings present within each pixel, including those between leaves, combine their effects to reveal the primary pixel-sized earth radiator, which is now much larger, and the mineral signature predominates.

Moisture Factor

The single greatest deterrent to acquisition of usable imagery is surface moisture. Moisture absorbs heat from the soil and effectively destroys the mineral spectral signature. Several image acquisition flights, in 1984, 1989, and 1990, suffered from rainfall up to 4 days before the flight, resulting in imagery that is useless for gravel deposit inventory. Surface moisture completely alters the nature of the image, and shows only patterns of vegetation. The spectral signature immediately following a period of rainfall is relatively uniform over the entire image, with low points in Bands 1, 4, and 6; high points in Bands 2, 3, and 5, assuming the shape of saw teeth.

NASA INVOLVEMENT

NASA's scientific interests are in determining how the thermodynamic responses of surface minerals contribute to the overall spectral radiant emittance in the thermal infrared spectrum. This understanding will contribute greatly to extending the use of such imagery for natural environmental applications. A major objective of this cooperative supporting study is to determine under what physical limitations and restrictions thermal infrared imagery can be used to discriminate gravels from surrounding materials in a variety of environ-

mental settings. This will be accomplished through extensive empirical measurement and modeling of the radiometric response over several regimes of soil moisture, humidity, vegetation, and atmospheric condition.

During FYs 1989 and 1990, NASA focused its efforts on conducting controlled experiments to determine when the differential thermal response of surface materials such as gravels, sands, and clays with inherently differing thermodynamic properties peaks during annual and diurnal heating cycles, exhibiting high contrast in thermal images. This information will be used to program data acquisitions during periods of the greatest temperature differentials between targeted gravel deposits and surrounding background materials.

In order to accomplish this objective, a set of remote instrument stations were designed and engineered to monitor surface heat flow parameters, and installed at STL for testing, both at a graveled and a nongraveled control site. These modified micrometeorological stations measure the following properties: incoming and outgoing short- and long-wave radiation (0.3 to 50 μm); net radiation; air, surface, and subsurface temperatures at three depths; wind speed; relative humidity; soil moisture; and conductivity. From these measurements, heating and cooling response can be calculated for the materials present, using known models to predict intervals of maximum temperature differences. Figure 4 shows a block diagram and schematic of system components.

A known gravel deposit at the new Black Creek Seed Orchard at the W. W. Ashe Nursery on the Desoto National Forest in southeastern Mississippi was selected because of its proximity to STL. Exploratory drilling with the Forest Service's drill rig revealed a half-million m^3 of gravel deposits over the surface, 5 to 8 m in thickness. The micrometeorological stations were transported to the site and installed within the gravel deposit, and in an adjacent ungravelled silty soil, in early July 1989. A telemetry connection to STL for data monitoring has since been installed. During September 1990, two additional stations with telemetry were installed on the Kisatchie National Forest in Louisiana following a period of testing at STL.

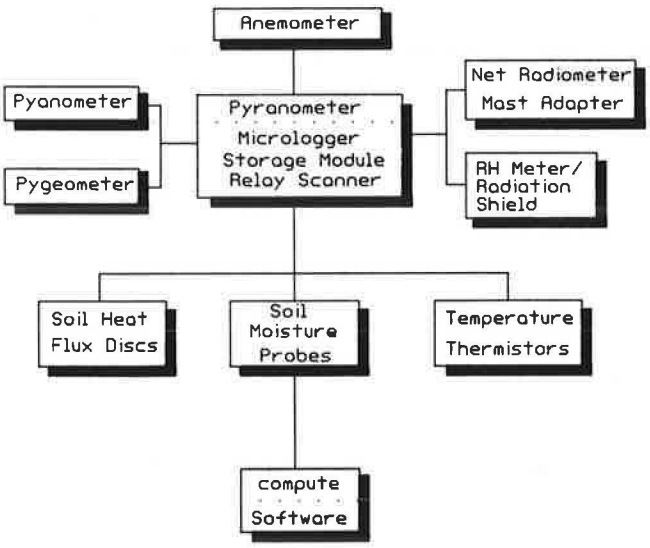

FIGURE 4 A block diagram of NASA's micrometeorological station installed on the Desoto National Forest.

From the initial month of data recorded by the stations installed on the Desoto National Forest, it was obvious that the largest temperature differentials were caused by the summer solar insolation maxima and a brief lag time for ground warming caused by thermal inertia. It also became apparent that the gravels heated and cooled more rapidly over the diurnal cycle than the deposit of finer-grained sediments. Summer data show the gravels to be up to 14°C warmer during the day with the differential peak occurring around 2:30 p.m. The corresponding predawn differential peaks at up to 4°C cooler for the gravels at 6:30 a.m.

In order to provide a correlation between ground and aircraft data, three missions of three lines each were flown with the calibrated airborne multispectral scanner (CAMS). The missions were flown at the times stated on the afternoon of August 18, and during the predawn hours and the afternoon of August 19 to catch the peaks of the diurnal cycle. The CAMS is an internally calibrated scanning spectroradiometer with eight bands in the visible and reflective near-infrared spectrum and one wide thermal infrared band. Images have been geometrically rectified to UTM coordinates, reduced to temperature differences, and processed into actual differences of thermal inertia present at those points in time. The processed values agree closely with values calculated from in situ measurements.

FUTURE EFFORTS

Although a good preliminary understanding of the TIMS imagery for the Kisatchie National Forest has been attained during the past 2 years of effort, much remains to be done. The procedures developed using the initial data set must be tested with imagery of the same locations in other years, other times of the year, and at other locations with differing soil cover. Soil monitoring data must be combined with weather station data to determine optimum time or times during the year for obtaining the best imagery. The use of data combinations from two or more times of the year for the same location should be studied. Efforts should be made to develop a better understanding of the physics involved.

Work currently underway on and around the Ashe Nursery on the Desoto National Forest in Mississippi offers the first opportunity to pursue these objectives. This site provides all the components required for such a study: imagery, ground data, and known gravel deposits.

Additional sites under consideration include the Sam Houston and Davy Crockett National Forests in east Texas, and reflights of the Kisatchie National Forest in Louisiana. Other areas under consideration include national forest land in Arizona, Colorado, and Idaho. This list of potential sites will probably expand as the study progresses.

CONCLUSIONS

Significant progress has been achieved in understanding the uses of TIMS imagery for gravel deposit exploration on the Kisatchie National Forest in Louisiana. Tentative gravel signatures have been identified that provided reliable results in field tests performed during 1989 and 1990. These investi-

gations have resulted in the discovery of 1 million m^3 of gravel deposits. Data developed solely from the TIMS imagery has successfully been used to produce a tentative geographic information system gravel deposit inventory map for selected areas of the Kisatchie National Forest.

The information developed can be used as a basis for further study into the means of using TIMS imagery for gravel exploration in a broad variety of environments.

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Shallow Refraction Surveys on a Low-Volume Road for Determining P-Wave Velocity of Seasonal Thawing Soils

B. D. ALKIRE AND C. KELLER

Determination of soil strength during spring thaw is essential to developing mechanistically determined seasonal load restrictions. The work reported uses shallow refraction techniques to determine P-wave velocities of the road subgrade at weekly intervals. From this information, the variation of P-wave properties with time is developed. From the relationship between P-wave velocity and time, it is shown that P-wave velocity starts at relatively high values before thaw commences, decreases to a minimum, and then increases. The minimum P-wave velocity occurs at about 3 weeks after the maximum degree-days for the freezing season. Results from the P-wave tests are shown to be comparable to results obtained using a Clegg impact test. However, the minimum Clegg value occurs at an earlier date than the minimum P-wave velocity. Overall, results indicate that shallow refraction surveys can be used to develop a curve of P-wave velocity versus time. This relationship, in turn, can be related to a soil strength value that would be usable in a mechanistic pavement design method.

The timing of road restrictions during spring breakup is one that has plagued many state, county, and city highway officials. Presently, the decision of when to impose and lift load restrictions is left to the judgment of the road commission or county engineer. They base their decisions on past performance of the roadways during the spring thaw, visual evaluation of the roadway's strength, and, possibly, measurement of some soil property. In most cases, the methods of determining when to place and lift load restrictions is subjective; however, if the timing is not correct, there can be economic losses to the transportation community.

In this study, the seismic refraction technique is used to determine the P-wave velocities of both frozen and unfrozen subgrade materials. Then, using methods from wave theory, the velocities are related to the soil's elastic properties (one of the physical properties that changes on freezing and thawing). By repeating the test at regular intervals during spring breakup, the seismic method can be used to detect changes in the road strength. This, in turn, can be used to make rational decisions about when to apply and lift road restrictions.

The tests are from a series of shallow refraction surveys conducted at a test site near Houghton, Michigan. Tests were conducted on a regular basis throughout the spring thaw periods of 1986, 1987, and 1988. At the same time and site, Clegg impact tests were conducted to provide correlation with another indirect method of assessing soil stiffness. Because of the nature of the test site, the results are appropriate to aggregate-surfaced roads only.

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TEST SITE

The road chosen for this study was a low-volume aggregate-covered road south of Houghton, Michigan. The wearing surface of the road is a 5-in. (127-mm) layer of a local aggregate known as "stamp sand" with a USCS designation of SW–SM. The naturally occurring subgrade is a silty sand with 26 percent of the material finer than a #200 sieve and a liquid limit of 17. The soil has a frost susceptibility designation of F4. The subgrade soil is extremely frost susceptible and contains microscopic excess ice. During spring breakup, the road experiences severe distress in the form of rutting, frost boils, cracking, and potholes.

FIELD TEST

Seismic compressional wave (P-wave) velocities of the subgrade of the road were found using typical shallow seismic refraction techniques. For the tests reported, single-ended hammer seismic refraction surveys with a spread length of 50 ft (15.2 m) were run using a Nimbus 55–125 single-channel, signal enhancement seismograph. For a typical test, the geophone was installed at the zero station and a tape was used to lay out the hammer blow locations at regular intervals along the survey line. Successive hammer impacts on a strike plate were repeated until a well-defined first-arrival wave was observed on the instrument's cathode ray tube. The travel time for the first arrival was noted, and the impact source was moved to the next station, where the process was repeated. The geophone was always set at the same location and spreads were run parallel and transverse to the centerline of the road. The short spread length, shallow depths probed, and nature of the soil layering justified use of a single-ended survey.

Early results indicated that the parallel to centerline profile was probably more typical of the assumed horizontal layer boundary condition because of the fact that thaw depth was nearly constant along any line parallel to the centerline. However, across a transverse section, thaw depth was greater near the centerline and decreased with distance toward the shoulder. As a consequence, only results from the centerline profile are used in the analysis.

The condition of the road at the test site was also measured using a Clegg impact test. This apparatus was developed in

Australia in the mid-1970s as an in-place stiffness test (*1*). The instrument used in this research project had a 10-lb (4.54-kg) hammer and was dropped 18 in. (457 mm). The Clegg impact value (CIV) is the reading obtained on the recording meter after the fourth drop.

RESULTS

For each field test, the distance from the source to the geophone and the time for the P-wave first arrival was obtained. A typical data sheet (for the test on March 12, 1986) is shown in Figure 1. On this sheet are the measured distance from impact source to geophone, the observed first-wave travel time to the geophone, and the CIV value of the soil taken at a position adjacent to the strike plate. Also shown on the figure are the time, date, and weather conditions at the site, as well as the calculated average CIV values and standard deviations. Data from tests were used to plot graphs of distance from geophone versus travel time of first arrival. Figures 2–4 show typical results obtained at the same location on three different days.

Figure 2 is typical of the results obtained for an unfrozen soil without a well-defined layer to refract the P-wave. The test was conducted on April 28, 1988, and the velocity associated with this curve is approximately 2,000 ft/sec (610 m/sec), typical of the P-wave velocity in a thawed silty sand with a relatively low water content. The actual measured water content 3 ft (0.91 m) below the surface on this date was 10.5 percent.

When there is a low-velocity layer over a higher-velocity layer, the normal relation between time and distance occurs as shown in Figure 3. In this case, the top layer is the thawed subgrade soil with a high water content and low velocity of

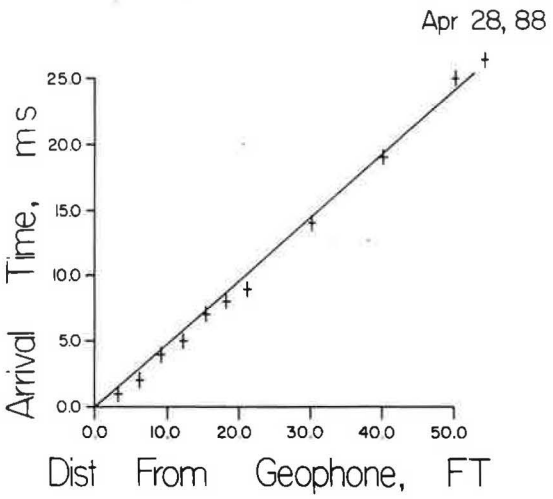

FIGURE 2 Typical arrival time versus distance for unfrozen subgrade soil.

1,773 ft/sec (540 m/sec) and is over the frozen subgrade soil with the higher velocity of 4,433 ft/sec (1351 m/sec). The relatively short distance to the break in the curve is caused by the shallow depth of the top layer. For example, if the break in Figure 3 is at 6 ft (1.83 m) and the time intercept is at 2 msec, the thickness of the thawed layer can be calculated using the formula (*2*):

$$T_1 = 0.5t_i V_1 V_2/(V_2^2 - V_1^2)^{1/2} \quad (1)$$

where T_1 = thickness of Layer 1, V_1 = velocity of P-wave in Layer 1, V_2 = velocity of P-wave in Layer 2, and t_i = time intercept. The calculated value using the values obtained from

Massie Rd. Project Data: 3-12-86; Weather: T = 35F, Sunny

Note: Thaw about 3" into road surface, heavy rutting

Profile	Distance (ft)	Time (ms)	CIV
Centerline	0.00	0.00	
	3.00	0.20	48
	6.00	0.40	55
	9.00	0.60	48
	12.00	0.70	45
	15.00	1.20	51
	18.00	1.90	41
	21.00	2.50	49
		Avg. CIV	48
		Std. Dev.	4.1
Transverse	0.00	0.00	34
	4.00	0.10	33
	6.00	0.30	35
	8.00	0.50	34
	10.00	0.80	32
	14.00	1.30	65
	18.00	1.70	
	21.00	2.50	
		Avg. CIV	39
		Std. Dev.	11.7

1 ft. = 304.8 mm

FIGURE 1 Typical data sheet for test program.

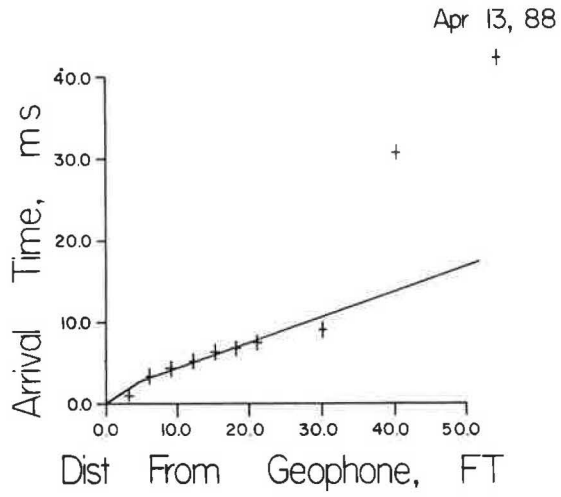

FIGURE 3 Typical arrival time versus distance for partially thawed subgrade soil.

Figure 3 is 23 in. (584 mm). This value is close to the observed 20-in. (508-mm) depth of thawed soil.

The most interesting and hardest relationship to explain is shown in Figure 4. The apparent velocity reversals shown here should not be obtained from a normal refraction survey. However, Irving (*3*) explains that reverse breaks of this type result from rapid attenuation of first arrivals and are common in frozen ground. The fact that the signals were being attenuated rapidly manifested itself in this test series as the amplifier gain had to be increased to nearly its maximum value when the energy source was at 50 ft (15.2 m). Another factor that contributes to this behavior is the change in Poisson's ratio that occurs as a soil goes from frozen to thawed. The velocity of the frozen soil is determined from the steepest slope and for this case is approximately 4,500 ft/sec (1371 m/sec).

At the same location used for the hammer blows for the P-wave test, Clegg impact tests were also conducted. It has been shown that the Clegg values follow a thaw recovery curve (*4*). For the tests conducted as part of this work, the Clegg value was obtained at several locations along the survey line,

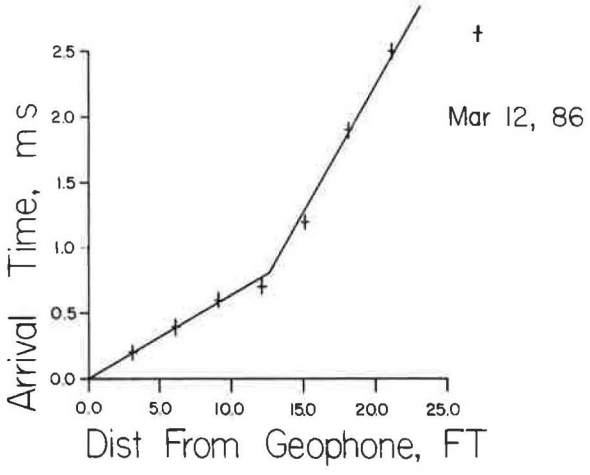

FIGURE 4 Typical arrival time versus distance for frozen subgrade soil.

and the average value for a particular test was calculated as shown in Figure 1.

A summary of all test data is presented in Table 1. This table lists the date of each test, P-wave velocity, and the average CIV value for the centerline profile on that particular date. Also presented in this table is the number of days (+ or −) that the test date was when compared to the day when the maximum value of freezing Fahrenheit degree-days occurred (this is assumed to be the day when spring-thaw begins). Note that both the P-wave velocity value and the CIV values start high, decrease to a minimum, and then increase. In general, the minimum P-wave velocity and CIV value occurred at some time after the day of maximum freezing degree-days.

DISCUSSION OF RESULTS

Results from the study indicate that it is possible to use the velocity of the direct P-waves to monitor the roadway condition through spring breakup. Figure 5 shows a plot of direct P-wave velocity versus number of days since the start of thaw for the three seasons of data. It can be observed that the curves have similar shapes but are offset slightly from year to year.

An empirical equation relating P-wave velocity and days since the beginning of thaw is also plotted on Figure 5 and was developed from the following equation:

$$V = 2784.0 - 152.4T + 3.0T^2 \quad (2)$$

where V = P-wave velocity in ft/sec and T = number of days since thawing weather began. The correlation coefficient for the equation is 0.85, and indicates that time since thaw began is a good predictor of P-wave velocity.

Figure 5 and Equation 2 demonstrate that soil velocity varies substantially. As would be expected, 1 to 2 weeks before the start of thaw the soil is frozen and the velocity can be greater than 6,000 ft/sec (1824 m/sec). As the average daily temperatures increase, the velocity decreases to a minimum around 20 days after the start of thaw. Finally, as thawing degree-days continue to accumulate, the velocities increase until they are again at relatively high values 30 to 50 days after the beginning of thaw. If Equation 2 is differentiated with respect to time, the minimum value of the velocity is found to occur at 22 days after thaw begins. The resulting value of the velocity obtained using Equation 2 is 883 ft/sec (269 m/sec).

From the known (*2*) relationship between the compression wave velocity and the modulus of elasticity, it is possible to calculate the modulus of elasticity:

$$E = V^2p(1 + \mu)(1 - 2\mu)/(1 - \mu) \quad (3)$$

where E = modulus of elasticity, V = P-wave velocity, p = mass density of the soil, and μ = Poisson's ratio. Using Equation 3 and a compression wave velocity of 1,000 ft/sec (304 m/sec) results in a modulus of approximately 6,800 psi (46.9 MPa) if it is assumed that Poisson's ratio is 0.45 [typical for frozen soil near 32°F (*5*)] and the unit weight of the soil is 120 lb/ft^3 (18.2 kN/m^3).

TABLE 1 SUMMARY OF TEST RESULTS

Date	P-Wave Velocity (fps)	Average Clegg Impact Value (CIV)	Days Before (-) or After (+) day of Maximum Freezing Fahrenheit Degree-day (days)
March 18, 1986	5822	26	- 7
25	NA	34	0
April 1, 1986	1272	13	7
7	1385	18	+13
15	982	10	+21
22	1041	30	+28
30	1261	36	+36
May 6, 1986	1705	43	+42
March 16, 1987	5199	37	- 2
20	1477	42	2
24	1134	28	6
April 6, 1987	799	40	19
19	1685	40	32
30	3000	47	43
March 17, 1988	5118	190	-16
24	4623	28	- 9
31	3019	33	- 2
April 5, 1988	998	12	3
7	962	19	5
13	1773	39	11
28	2239	48	16

1 fps = .304 mps

The modulus obtained using Equations 2 and 3 could be used in any road design technique that requires this parameter. For example, a simple rut depth formula in the form of a uniaxial stress-strain relationship might be the following:

$$R = KW/E \tag{4}$$

where R = rut depth, W = wheel load, E = modulus of elasticity, and K = a constant. By assuming a limiting rut depth, the minimum modulus and related times when they occur could be calculated and the dates (±days after beginning of thaw) the load limits should be applied and lifted would be determined. Obviously, once the relationship between modulus and time is known, it would be relatively easy to set a load restriction based on even more sophisticated techniques such as those proposed by the U.S. Forest Service (*6*) and AASHTO (*7*).

In order to correlate the results for P-wave velocity with another test, there was an attempt to do a regression on the CIV value versus days after thaw begins. The results are plotted in Figure 6 and the regression equation developed from the data presented in Table 1 is

$$\text{CIV} = 29.0 - 0.48T + 0.020T^2 \tag{5}$$

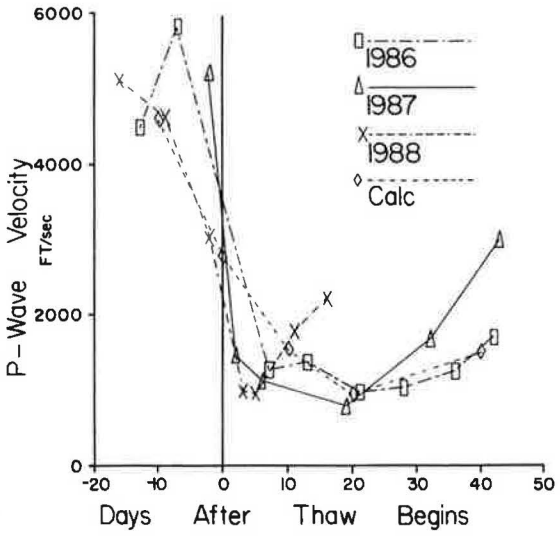

FIGURE 5 Summary P-wave velocity versus days after thaw begins.

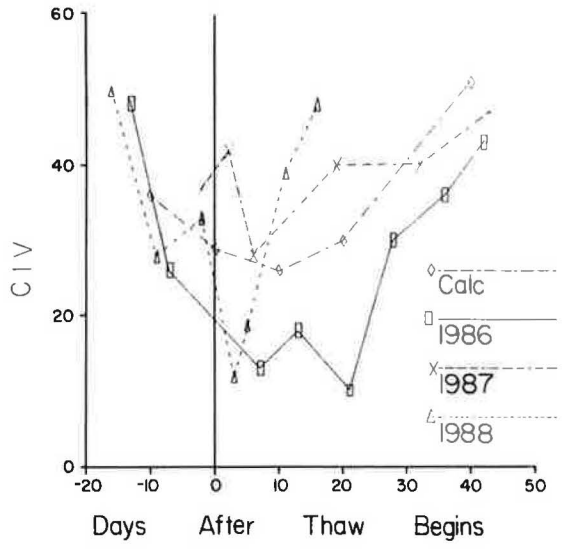

FIGURE 6 Summary CIV versus days after thaw begins.

with a correlation coefficient of 0.52. The low correlation coefficient for this equation indicated that Equation 2 is a better predictor of P-wave velocity than Equation 5 is of CIV. This should be expected because the Clegg device provides a measure of the near-surface characteristics of the soil and thus is more susceptible to small changes in the surface of the road caused by daily changes in the ambient temperature.

By differentiating Equation 5, the minimum value of CIV is determined to occur at +7 days after thaw begins, which is almost 2 weeks before the minimum date predicted by the P-wave velocity equation.

Because the regression coefficient for Equation 2 is higher than for Equation 5, the P-wave velocity may be a more reliable predictor of soil strength. Thus, the seismic refraction technique may provide a more reliable way of developing a thaw strength recovery curve and predicting when load restrictions should be applied on low-volume roads.

CONCLUSION

It is possible to obtain an indication of the thaw recovery curve by conducting shallow seismic refraction tests on a regular basis. On the basis of the results, the following observations can be made:

1. P-wave direct arrival velocity decreases as the soil goes from the frozen to thawed state.
2. The minimum value of P-wave velocity will occur at some time well after the day when the freezing degree-day curve has its maximum value.
3. Clegg impact values are more sensitive to minor changes in ambient temperature conditions than results from a seismic refraction test. Although the CIV values decrease to a minimum and then increase in much the same way as the P-wave velocity, individual reading are more likely to have large variabilities.
4. P-wave velocity can be used to circulate the modulus of elasticity of the surface layer and could be used as a direct indication of the soils stiffness.

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Materials and Stabilization

Use of Cold In-Place Recycling on Low-Volume Roads

TODD V. SCHOLZ, R. GARY HICKS, DAVID F. ROGGE, AND DALE ALLEN

In the last several years (since 1984) Oregon has used cold in-place recycling (CIR) techniques as one alternative to conventional practices for the rehabilitation of asphalt concrete (AC) pavements. All CIR efforts have consisted of partial-depth (AC-surface) recycling on low-volume roads—generally less than 2,000 average daily traffic—with good success on most projects. Projects have also been completed on county roads and U.S. Forest Service roads using this technique. In most cases, the recycled mat is treated with a sand seal or a chip seal. Construction costs for this operation are on the order of $1.70 to $2.10 per square yard compared with $2.75 to $4.00 per square yard for a conventional 2-in. AC overlay. Because of the success of the initial projects that were cold recycled in 1984 to 1985, the Oregon State Highway Division (OSHD) and Oregon State University (OSU) have studied cold in-place recycling in detail since 1986. One purpose of the studies by OSHD and OSU has been to develop a better understanding of the performance and economics of the cold in-place recycled pavements. A brief overview of the CIR construction and design process, the performance to date of selected projects, and an economic evaluation of CIR as an alternative to other rehabilitation techniques are provided. Significant findings include (a) the current state-of-the-art in recycling equipment allows for efficient and economic recycling practices; (b) over 75 percent of the CIR projects in Oregon since 1984 were rated fair or better in 1984 to 1989; and (c) CIR can provide significant savings realized through conservation of energy and costly construction materials. On the basis of the results of the joint study to date, it appears that CIR is a viable rehabilitation alternative for low-volume roads. Hence, using the CIR concept on higher-volume roads (including Interstates) is now proposed.

With the national trend away from new construction to preservation of existing pavements, several agencies are turning to cold in-place recycling (CIR) as an approach to rehabilitating distressed pavements. However, many agencies remain skeptical of the use of CIR because of the lack of long-term performance data and adequately documented field engineering studies. Furthermore, because of variability in construction processes with substantially different design concepts and end results (*1–3*), the term CIR is often misunderstood.

Recycling may be defined as the reuse, after processing, of a material that has already served its intended purpose. The different construction processes for cold in-place recycling are defined as follows:

1. Class I. This recycling treatment is performed on a uniform pavement designed and built to specifications. It is expected that a rational CIR mix design can be prepared and produced. The treatment could handle medium-to-heavy traffic volumes, usually as a base on high-volume roads or as a wearing course on low-volume roads. The recycling train method would normally be used; however, depending on the degree of distress, a single-unit train could also produce a Class I treatment. Treatment width is normally 12 ft.
2. Class II. This recycling treatment is performed on a pavement with significant maintenance patches over a uniform pavement or a pavement with minimal design used in the original construction. Either the recycling train or the single-unit train can produce millings of sufficient quality for reasonable mix designs. The finished mix can be used as a base or wearing course as in the case of the Class I process. Treatment width is normally 12 ft.
3. Class III. This treatment is used on low-volume highways where considerable variation in pavement structure exists and it may incorporate additional aggregate. Various milling and pulverizing units can be used to perform this operation. The treatment is normally used as a stabilized base course. Treatment width varies from 4 to 12 ft.

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The Oregon State Highway Division is one of the several agencies that has attempted CIR as an approach to rehabilitating distressed asphalt concrete (AC) pavements. Oregon first experimented with partial-depth CIR work in 1984, totaling 14 mi. An additional 68 mi of AC pavement was cold recycled in 1985. Spurred by the initial success of these projects and recognition of the need for a formal mix design procedure, ODOT and OSU, in 1986, undertook a joint study of CIR. The study involved investigating 7 of 13 projects cold-recycled in 1986 to develop an improved understanding of the relationship between mix design and field performance of cold-recycled pavements. The specific objectives of the study were to develop an improved mix design, evaluate the structural contribution and durability, and develop improved construction guidelines and specifications for CIR pavements.

The history of CIR in Oregon from 1984 to 1989, including project information and the construction process used on the projects; the evolution of the design process for CIR; performance information for all projects constructed since 1984; and a life cycle cost analysis comparing CIR and hot mix are described. Also presented are significant conclusions from the work completed to date as well as recommendations for implementation of the findings to date.

HISTORY OF COLD RECYCLING IN OREGON

Project information associated with the 1984 to 1989 CIR work as well as the process used to construct the projects are described in the following subsections.

Project Information

To date, 500 mi have been cold-recycled in Oregon since 1984. All projects were constructed in Regions 2, 3, 4, and 5. Table 1 presents construction information for all of the Region 4 projects totaling 420 mi constructed between 1984 and 1989. Information for the projects constructed in Regions 2, 3, and 5 was unavailable at the time of writing.

Construction Process

Oregon's first efforts (1984) at CIR involved exclusively Class III recycling. The construction process was accomplished with a roto-mill having a 6.5-ft milling head and a motor grader. The surface was milled with the roto-mill, which discharged the millings into a windrow to the side of the cut. Water and CMS–2S emulsion were then applied to the windrow. The

TABLE 1 PROJECTS CONSTRUCTED IN OREGON (1984 to 1988)

Year	Highway	Project Name	Traffic Volume (ADT)	Length (mi)	Depth of Cut (in.)	Emulsion Type (Content)	Class of Treatment	Surface Treatment
1984	OR 372	Sand Shed-Mt. Bachelor (Intermittent)	820	4.8	1.5	CMS-2S (1-2%)	Class III, State forces, grader laid	Surface left open winter of 1984,
	Misc.	Bend area	Up to 2000	9.0	1.5	CMS-2S (1-2%)	Class III, State forces, grader laid	About 50% chip sealed
1985	US 26	Sisters-Redmond	1450-8300	18.8	1.5	CMS-2S (1-2%)	Class II	Chip seal placed on about 75% of work
	US 395	Harney Co. Line-Hogback Summit	220	30.7	1.5-2	CMS-2S[1] (1-2%)	Class I	Chip seal
	US 140	Drews Gap-Lakeview	1000	10.3	1.5-2	CMS-2S (1-2%)	Class I	Polymer chip sealed
	Misc.	Bend area	Up to 23,000	12.0	1.5-2	CMS-2S (1-2%)	Class I	80% chip sealed
1986	US 26	Warm Springs	2850	17.3	2-4	CMS-2S (1%)	Class I	Polymer chip sealed
	OR 41	Powell Butte-Prineville	3600	9.8	2	CMS-2S (1.2%) HFE-150 (1.2%)	Class I	Chip seal
	OR 270	Lake of the Woods	1750	6.4	2.5-4	CMS-2S (1.4%)	Class I	Chip seal
	US 20	Bend-Powell Butte	4800	3.2	1.5-2	CMS-2S (1.5%)	Class I	Chip seal
	OR 371	MP 18.0-Powell Butte	2200	18.0	1.5-2	CMS-2S (1.1-1.3%)	Class II	Chip seal
	US 26	Ochoco Dam-MP 35.0	1100	10.6	1.5-2	CMS-2S (1.1-1.6%)	Class II	Chip seal
	US 26	MP 73.4-MP 81.6	600	8.2	1.5-2	CMS-2S (1.8-2.6%)	Class II	3-in. overlay
	OR 41	MP 89.6-Jct. OR 19	600	8.7	1.5-2	CMS-2S (1.4-1.5%)	Class I	Chip seal
	US 20	MP 75.0-MP 84.0	1000	9.0	1.5-2	CMS-2S (1.5-1.6%)	Class I	3/4-in. oil mat
	OR 423	US 97-OR 39	800	7.0	1.5-2	CMS-2S (1.5%)	Class I	Chip seal
	OR 140	Dairy-Ritter Rd.	2000	6.0	1.5-2	CMS-2S (1.2-1.9%)	Class II	Chip seal
	OR 140	Sprague River Rd.-Bly	2700	17.8	1.5-2	CMS-2S (1.5%)	Class II	Chip seal

TABLE 1 *(continued on next page)*

TABLE 1 *(continued)*

Year	Highway	Project Name	Traffic Volume (ADT)	Length (mi)	Depth of Cut (in.)	Emulsion Type (Content)	Class of Treatment	Surface Treatment
1986	US 97	MP 235.3-Spring Creek	3400	6.0	1.5-2	CMS-2S (0.9%)	Class I	Chip seal
	OR 7	W-Horse Ridge-Crooked River Hwy	900	9.2	3	CMS-2S (1.7%)	Class I	Chip seal
	OR 372	Kiwa Springs-Sand Shed	880	5.6	2	CMS-2S (1.0%)	Class I	Chip seal
1987	OR 41	Antone-MP 89.6	520	8.0	1.5	CMS-2S (1.7%)	Class I	Chip seal
	OR 293	Jct. US 97-Tub Springs Rd.	200	9.0	2	CMS-2S (2.8%)	Class I	Chip seal
	OR 360	Jct. US 97-SE Rammes Rd.	1000	9.0	2	CMS-2S (1.6%)	Class I	Chip seal
	OR 380	Conant Basin Rd.-Shotgun Rd.	180	9.1	2	CMS-2S (1.0%)	Class I	Chip seal
	OR 4	Fuego Rd.-Forge Rd.	3350	9.9	2	CMS-2S (1.2%)	Class I	None
	OR 427	Modoc Secondary	450	11.2	1	CMS-2S (1.3%)	Class I	Chip seal
1988	OR 4	Shaniko Jct.-Quaale Rd.	390	12.8	2	HFE-150 (0.8-1.2%)	Class I	3/4 in. cold mix overlay
	OR 41	Prineville-Ochoco Dam	2200	7.0	2	HFE-150 (1.8%)	Class I	†
	OR 41	Ochoco Ranger Sta.-Ruch Creek	800	19.7	2	HFE-150 (0.6%)	Class I	Chip seal
	OR 380	Jct. Ochoco Hwy-Conant Basin Rd.	3100	20.7	2	HFE-150 (2.6%)	Class I	†
	OR 50	Merill Jct.-Hatfield Hwy	3600	2.6	2	CMS-2S (1.2%)	Class I	None
	OR 426	Jct. Klamath Falls-Malin Hwy to Calif. Line	2350	3.0	2	CMS-2S (0.5%)	Class I	None
	OR 42	DeMoss Springs-Moro	1800	5.0	2	HFE-150 (1.0%)	Class I	Chip sealed with HFE
	OR 19	Cogswell Creek-New Pine Creek	600	5.7	2.25	CMS-2S (1.7%)	Class I	Chip sealed with HFE
	OR 20[2]	Beatty-Ivory Pine Rd.	980	9.5	2.25	CMS-2S (0.3%)	Class I	None
	OR 22	Fort Klamath-Crooked Creek	550	5.4	2	HFE-150 (1.1%)	Class I	Sand seal

TABLE 1 *(continued on next page)*

TABLE 1 *(continued)*

Year	Highway	Project Name	Traffic Volume (ADT)	Length (mi)	Depth of Cut (in.)	Emulsion Type (Content)	Class of Treatment	Surface Treatment
1988	OR 49	Lake Abert-Valley Falls	260	4.0	2	CMS-2S (1.0%)	Class I	Chip seal
	OR 22	Crater Lake Hwy-Frontage Rd.	520	3.5	2	CMS-2S (1.7%)	Class I	Chip seal
1989	OR 4	Gilchrist Section	3400	1.0	2	CMS-2S (0.5%) (w/lime)	Class I	Sand seal
	OR 7	Horse Ridge-Crooked River Jct.	900	7.4	2	HFE-300S	Class I	Cold mix overlay
	OR 425	Umpqua Jct.-US 97	†	13.7	2	CMS-2S	Class I	Chip seal

[1]HFE-150 and HFE-150S were also used, but only for test
[2]One lane only
†Not Available

windrow was then mixed with the motor graders and bladed into the cut.

All subsequent work (1985 to 1989) was accomplished using either the recycling train or a single-unit machine. The work done with the recycling train was contracted out to a construction company that owned the equipment and most of their work would be classified as Class I or Class II treatments. The Oregon DOT maintenance team, on the other hand, relied on the use of a single-unit machine (Class I or Class II treatments). Both construction methods are discussed.

Recycling Train

In the train method, the train was led by a water tanker, and then a CMI 1000 roto-mill having a 12.5-ft milling head. The mill pulled a trailer-mounted screen deck, roll crusher, and pugmill followed by a nurse tanker for the emulsion.

The existing pavements were milled using the CMI 1000 to depths between 1.5 and 2.25 in. The millings were transferred by conveyor belt to the screen deck and screened over 1.5- to 2-in. screens. The oversized millings were crushed such that 100 percent passed the 2-in. screen. Emulsion (CMS–2S or HFE–150) was added and mixed with the millings in the pugmill. This mixture was deposited in a windrow on the roadway. A diluted CMS–2S tack was applied to the milled surface using a spray bar attached to the rear of the train. The train has controls to monitor quantity of emulsion and water.

In order to avoid difficulties in handling of the mixture, the paving machine was operated within 100 to 200 ft of the train. After laydown, a two-stage compaction was specified. The initial compaction was accomplished using a rolling pattern of one pass vibratory and one pass static with an Ingersoll Rand model DA–50 double-drum vibratory roller and one-pass static using a Hyster model 15–7 tandem steel wheel roller. The mat was opened to traffic immediately following initial compaction. The second-stage compaction followed within 3 to 12 days. The variation in days elapsed until second compaction is because of the amount of cure the recycled pavement has undergone, which depends primarily on pavement temperature and moisture content. That is, with high pavement temperatures and low moisture content, second compaction may be appropriate after only 3 days following pavement recycling, whereas up to 12 days may be appropriate for low pavement temperatures and high moisture contents following pavement recycling. The second compaction consisted of at least two passes of a Hyster 8-ton double-drum roller in static mode and at least two passes with a 20-ton pneumatic roller. The second-stage compaction is more effective than the initial compaction. This is because second-stage compaction results in a mat at the same (or nearly the same) density that exists in the wheel tracks that have been compacted under traffic since initial compaction. That is, the second-stage compactive effort merely levels the surface to match the compaction in the wheel tracks caused by traffic.

If humps or rough spots existed in the recycled mat after second compaction, they were removed with a milling machine or corrected with skin patches before sealing. Two weeks or more after recycling, the pavement was covered with a ⅜-in. × #10 single-chip seal [using a CRS–2 or a polymer modified (HFE–150) emulsion] or a fog/sand seal. Through experience, it has been found that a fog/sand seal is best for pavements with a relatively tight surface and having soft asphalt properties. A chip seal, on the other hand, would be appropriate for a cold-recycled mat with an open texture.

Single-Unit Train

The single-unit process involved use of a RAYGO Barco Mill 800. This unit has a 12.5-ft milling head and was serviced by a water and emulsion tanker. A modification was made to the unit to include a spray bar for applying tack immediately

ahead of the windrow. Placement was accomplished using a conventional paver. Initial compaction was the same as for the recycling train, but the second-stage compaction was normally done with only a vibratory roller because a 20-ton pneumatic roller was not available.

PROJECT DESIGN AND IMPLEMENTATION PROCESS

Implementation of CIR is basically a five-step process as follows:

1. Project selection,
2. Evaluation of candidate projects,
3. Mix and structural design,
4. Development of construction guidelines and specifications, and
5. Construction process including quality control.

Figure 1 shows the preconstruction steps in a flowchart and the following sections describe each implementation step in detail.

Project Selection

The applicability of cold recycling has been a source of some concern. It has been found through experience (*4,5*) that CIR is proving to be an effective treatment under certain pavement, climate, and traffic conditions. Table 2 presents some guidelines to determine whether or not CIR is applicable. Each of these conditions is described in detail in the following paragraphs.

Cold recycling should not be performed in areas that cannot accommodate the traffic volume during construction. That is, CIR is not recommended in areas that would result in excessive traffic control problems. CIR is also not recommended, at this time, for pavements that have exhibited stripping or pavements that have rutted because of an unstable fat mixture. Research is underway to evaluate whether or not CIR is an effective treatment for pavements that have stripped.

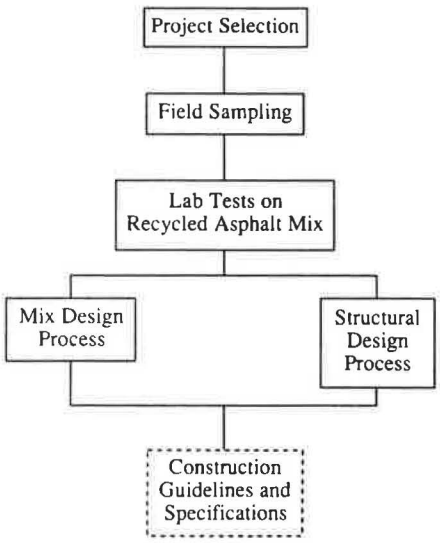

FIGURE 1 Preconstruction steps for CIR.

CIR work should not be performed in cold and damp conditions because these conditions inhibit the breaking and curing of the emulsion. Furthermore, CIR should not be performed in late fall or early winter, because the recycled pavement requires several days and nights of warm weather for proper curing.

Cold recycling is recommended for rough pavements, cracked and broken pavements, and pavements that have raveled because of age. CIR is effective in improving the ride quality of rough pavements and pavements that have raveled because of age. It is also effective in rehabilitating the structural integrity of cracked and broken pavements. CIR is also recommended for and can provide quality leveling of base courses for overlays. The constraint of ADT 5,000 or less is needed at the higher traffic level for safety and public inconvenience reasons only (i.e., excessive traffic control problems).

Evaluation of Candidate Projects

After a project has been identified as a candidate for CIR, a paper search on the history of the highway is performed. Type of asphalt, pavement thickness, and termini of previous jobs are the principal information to be obtained. On the basis of the available records or knowledge (Figure 2), the project is divided into design areas of uniform properties (e.g., pavement thickness and oil content). These are shown as *A*, *B*, and *C* in Figure 2b. Samples from each area should be obtained using a small 16-in. mill with the sample frequency being a minimum of 1 sample plus 1 backup sample per design area. For long sections, it is recommended that as many as three samples plus three backup samples should be obtained (Figure 2c). Samples should weigh 100 lb each to ensure adequate amounts of material are available.

The sample locations should be selected by visual inspection that identifies representative locations within the design area. The depth of milling for samples should correspond to that of the proposed depth of recycling. If the design area contains visible maintenance patches or other intermittent treatments, samples should be obtained from these areas, noting on each sample that it came from a maintenance area. All samples should be kept separate (should not be blended) and submitted to the laboratory for testing.

Tests to be performed by the laboratory on the rap millings obtained from the field sampling should include the following:

1. Penetration at 77°F of Abson-recovered asphalt,
2. Absolute viscosity at 140°F of Abson-recovered asphalt,
3. Gradation of the rap millings (16-in. mill), and
4. Extracted asphalt content.

Values obtained from these tests are to be used to estimate the optimum design emulsion content.

Mix Design

The mix design procedure consists of estimating the design emulsion content and preparing and testing samples at the

TABLE 2 CONSIDERATIONS FOR PROJECT SELECTION

a) CIR Not Recommended
• Work area cannot accommodate traffic volume
• Asphalt is stripping from aggregate*
• Mixes exhibiting rutting due to unstable fat mixture
• Cold and damp conditions during and immediately after construction (i.e., a mat temperature of 90°F or greater for at least 2 hr after laydown and in the absence of rain)
• Late fall or early winter treatment
b) CIR Recommended
• Cracked and broken pavements
• Pavements ravelled due to age
• Rough pavements
• As leveling and base for overlays
• ADT 5000 or less
• Where selective rehabilitation is needed (e.g., in truck lane of 4-lane roadway)

*Emulsions contain effective antistrip agents. While it is not recommended at this time that CIR be used to correct pavements with stripping problems, CIR may prove to be an effective treatment.

estimated design emulsion content and at the estimated design ±0.4 percent. These procedures are described in detail in the following subsections.

Estimating Design Emulsion Content

Estimation of the design emulsion content begins with establishing a base emulsion content and making adjustments on the basis of the results of the laboratory findings. Oregon has found that a base emulsion content of 1.2 percent by dry weight of rap is a good starting point. This figure was determined by trial-and-error techniques; namely, recycling was attempted using emulsion contents of 1, 1.5, 2, 2.5, and 3 percent. It was found that recycled mats having emulsion contents of less than about 1.5 percent tended to ravel, whereas those having emulsion contents of about 2 percent or greater tended to rut. Thus, an emulsion content of 1.5 percent was established as a good starting point. Through substantial use of the estimation procedure described herein, this value was further refined to 1.2 percent in the 1987 construction season. Once established, adjustments are then made to this base content according to softness of extracted asphalt, gradation of the millings (16-in. mill), and the percent of recovered asphalt. The calculations to be made with the adjustments are as follows:

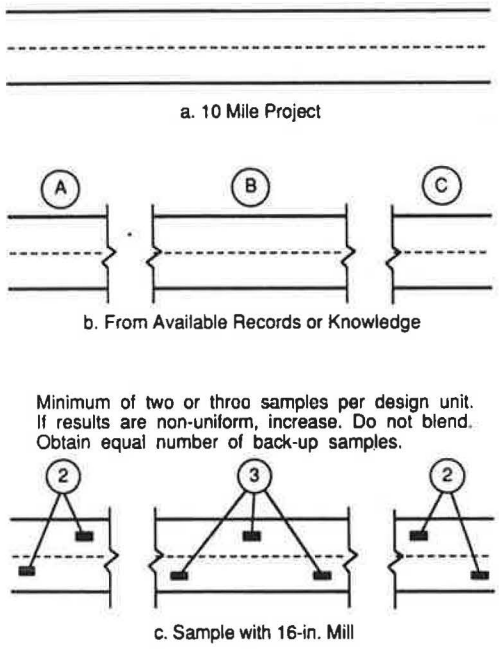

FIGURE 2 Suggested field sampling.

Base emulsion content	1.2 percent
Adjustment for softness	0 to +0.3 percent
Adjustment for gradation	±0.3 percent
Adjustment for percent asphalt	0 to −0.3 percent
Estimated design	
Lowest design	0.6 percent
Highest design	1.8 percent

The estimated design emulsion content can be as low as 0.6 percent and as high as 1.8 percent. This range represents the emulsion contents at which most projects are currently recycled; however, projects can be, and have been, successfully recycled with emulsion contents falling outside this range. The adjustments are discussed in detail as follows:

1. Softness of Asphalt. Penetration and absolute viscosity laboratory test results are used to determine the softness of the extracted asphalt. Figure 3 shows the ranges in these values that have been found in CIR completed to date. By plotting the values obtained from the laboratory on this figure,

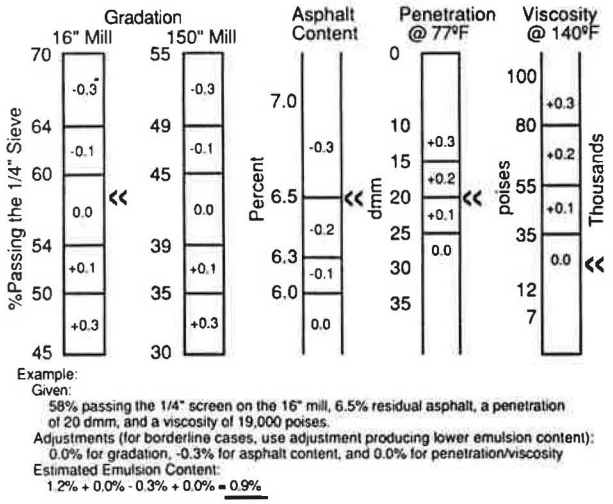

FIGURE 3 Emulsion content adjustments for gradation, asphalt content, and asphalt softness.

an adjustment of 0 to +0.3 percent can be selected. Thus, for a hard asphalt the emulsion content adjustment of up to 0.3 percent would be added to the base emulsion content. Conversely, no adjustment would be made to the base emulsion content for a soft asphalt.

2. Gradation Adjustment. By plotting the rap gradations from CIR completed to date, a range of values was obtained for the percent passing the ½-in., ¼-in., and #10 screens. Figure 3 indicates the range of values when the sampling is performed with a 16-in. mill and the expected rap gradation when using a 150-in. mill. By using this graph, a maximum adjustment of ±0.3 percent can be made to the base emulsion content. Rap with a coarse gradation would result in adding an adjustment of up to 0.3 percent to the base emulsion content, whereas up to 0.3 percent would be subtracted for rap with a fine gradation. This will seem intuitively incorrect in that traditional mix design procedures for hot-mix AC would prescribe increasing the asphalt content for fine mixes (relative to coarse mixes) to provide adequate coating of the fine aggregate particles because of the increased surface area of the aggregate. However, because the fines in rap millings appear to be predominately asphalt, and not aggregate particles, increasing the emulsion content tends to activate these fine particles of asphalt, resulting in an unstable mixture. Findings to date indicate that if a rap gradation is on the fine end of the range for the ½-in. screen, it will also be on the fine end for the range for the ¼-in. and #10 screens. The same holds true for a coarse or average gradation (*4,5*).

3. Asphalt Adjustment. The percent of asphalt recovered from the rap was plotted, giving the expected range of asphalt content. Figure 3 shows this range as well as the adjustment range of 0 to −0.3 percent. Rap with a high residual asphalt content would result in subtracting up to 0.3 percent from the base emulsion content, whereas no adjustment would be made for rap with a low residual asphalt content.

The estimated design emulsion content is determined as prescribed earlier. The significance of this procedure may be summarized as follows:

1. The procedure provides a rapid and simple method of determining emulsion content;
2. The laboratory tests used are widely accepted;
3. The procedure eliminates the necessity to fabricate, compact, and cure test briquets in the laboratory at simulated field conditions—one of the more controversial design issues for CIR (*4*);
4. The results generally produce the optimum emulsion content within a fraction of a percent; and
5. For most recycle projects in which preservation and restoration of an existing pavement are the primary objectives, the estimated design emulsion content would be adequate for the final recommended design.

Final Design

The final design emulsion content is determined from tests on samples prepared at the estimated design emulsion content and at the estimated design content ±0.4 percent. Figure 4 shows the steps for selecting a final design emulsion content when the CIR pavement will become part of the structural design to upgrade the surface. The samples should be prepared using either the Hveem or Marshall compaction method. Once compacted and cured as prescribed, the samples are tested for stability, resilient modulus, and index of retained modulus (IRM). The sample preparation procedure is as follows:

1. Split millings into approximately 5500-g batches; this size of sample provides sufficient material for four 6.4-cm (2.5-in.) specimens, with an 1100-g sample for moisture determination.
2. Screen samples on the 2.5-cm (1-in.) sieve. The material retained on the 2.5-cm sieve is reduced in size to 100 percent passing the 2.5-cm sieve using 13.4-N (3-lb) hammer. This is

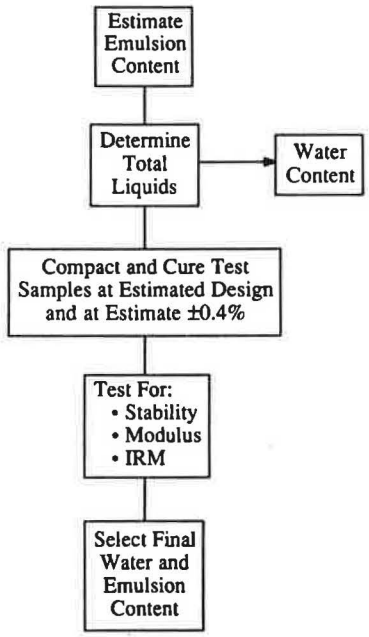

FIGURE 4 Suggested mix design process—future projects.

because the retained 2.5 cm is too large for 10.2-cm (4-in.) molds.

3. Batch five 1100-g samples of millings at the average gradation.

4. Determine moisture content of one batch by drying 24 hr at 100°C (230°F).

5. Heat samples to approximately 60°C (140°F) before mixing (1 to 2 hr).

6. Add water to the millings in the appropriate proportion based on the dry weight of the millings: percent water = 4.5 × total liquid − percent of added emulsion. Water is thoroughly mixed into millings by hand.

7. Add water to the premoistened millings after water addition using the recommended content. The added emulsion is based on the dry weight of the millings. The emulsion is preheated to approximately 60°C (140°F) for 1 hr and mixed thoroughly into the batch by hand or using a mechanized mixer.

8. Spread the material into a 30.5- × 41.2-cm (12- × 17-in.) baking pan and allowed to cure for 1 hr at approximately 60°C (140°F) to simulate average time elapsed between paver laydown and initial compaction during actual construction.

9. Mold samples using standard Marshall or Hveem procedures to produce 6.4-cm (2.5-in.) briquets as follows:

(a) Preheat molds to approximately 60°C (140°F).

(b) Compact samples using standard 50-blow compactive effort for Marshall procedure or 150 blows at 3.1 MPa (450 psi) for the Hveem procedure.

(c) Cure overnight at 60°C (140°F) and recompact using 25 blows per side for the Marshall procedure and 75 blows at 3.1 MPa (450 psi) for the Hveem procedure.

(d) Lay the molds on their sides and the briquets are cured for 24 hr at approximately 60°C (140°F) before extrusion.

(e) Extrude briquets with the compression testing machine.

(f) Lay briquets on their side to maximize surface exposure and cure them for 72 hr at approximately room temperature before testing.

10. Specimens are tested for stability, modulus, and fatigue at 25°C (77°F).

Structural Design

One of the objectives of this study was to develop structural layer coefficients for CIR mixtures. These coefficients would be used to determine the required thickness of the CIR pavement. As a result of the performance studies to date, it appears CIR mixtures may be considered essentially equivalent to conventional hot mix, as discussed in more detail in the next section.

FIELD PERFORMANCE (1984 to 1989)

Beginning in 1986, the following field and laboratory data were collected to evaluate the field performance of the CIR projects (*4,5*).

1. Pavement condition (visual surveys),
2. Ride (Mays meter), and
3. Mix properties (resilient modulus, fatigue, Marshall stability, and flow).

These data are summarized in the following sections.

Pavement Condition

Visual condition surveys were conducted on most of the projects in the falls of 1986 and 1987 as well as in the springs of 1988 and 1989. Pavements were rated on a scale of 1 to 5, with 1 being a condition rating of very good as prescribed by the Oregon State Highway Division's (OSHD's) rating procedure (*6*). A summary of the condition of selected projects (as of spring 1989) is presented in Table 3. As indicated, the projects with no mix design (1984) and those with the initial mix design (1985) are generally in fair condition. However, those with the modified mix design (1986) are performing with a fair to very good condition. Conventional mixes placed in the same geographical areas as the recycled pavement deteriorate at the same rate because of the extremely severe weather conditions that prevail in central to eastern Oregon.

Ride

Ride data were collected on several of the CIR projects using the Mays ride meter. Data were obtained immediately before and after the construction as well as each year after construction. These data are presented in Table 4. The criteria used to rate the smoothness of the pavement are as follows:

Mays Reading (in./mi)	*Rating*
200+	Very rough
150 to 200	Rough
100 to 150	Slightly rough
75 to 100	Average
0 to 75	Smooth

As indicated, ride was improved on the two projects that were rated rough before construction. For the two projects (Warm Springs and Lake of the Woods) that had a smooth rating before construction, the CIR work retained the ride rating.

Mix Properties

Field cores were extracted from several of the CIR projects beginning in 1986. Tests conducted on these cores were as follows:

1. Bulk specific gravity,
2. Resilient diametral modulus and fatigue [after Scholz (*7*)], and
3. Marshall stability and flow.

These data are presented in Table 5. All values represent the average of tests on three cores. Figure 5 graphically displays the modulus and fatigue results, and Figure 6 graphically displays the Marshall stability and flow results.

The modulus values increased with time for all sections. These increases were expected because of the additional cur-

TABLE 3 SPRING 1989 CONDITION

Year Built	Section (Note 1)	Hwy No.	M.P.	M.P.	Length (mi.)	Depth of CIR (in.)	Emulsion Used	Original Pavement (Note 2)	Rut Depth (in.) Lt.	Rut Depth (in.) Rt.	Thermal Crack Spacing (ft)	Flushing	Fatigue Cracks	Maint. Work	Rating	Notes
1984	Fremont Hwy-(N. Lane)	19	8.0	8.6	0.6	1-1/2	CMS-2S	C		1/16	20-60	--	Minor	Minor	Fair	Delete from study.
1984	Fremont Hwy-(N. Lane)	19	16.8	18.3	1.5	1-1/2	CMS-2S	C	3/16	5/16	20-60	Heavy	Minor	Major	Poor	Delete from study.
1984	Sand Shed-Mt. Bachelor	372	16.6	21.6	5.0	1-1/2	CMS-2S	C	1/16	3/16	occasional	Minor	Minor	Minor	Fair	Intermittent CIR between mile points.
1985	Sisters-Dry Creek	15	93.2	99.0	5.8	2	CMS-2S	C	1/8	1/16	30-50	Heavy	Minor	--	Poor-Fair	
1985	Dry Creek-Warrin Rd.	15	99.0	105.0	6.0	2	CMS-2S	O-11	1/8	3/16	30	--	Minor	--	Fair	
1985	Warrin Rd.-Redmond	15	105.0	111.9	6.9	2	CMS-2S	B	1/8	1/8	15-30	--	Minor	--	Fair-Good	
1985	Summit Drews Gap-Lakeview	20	81.8	92.8	11.0	1-1/2	CMS-2S		1/16	1/8	30-100	Minor	Minor	Minor	Fair-Good	
1985	Harney Co. Line-Bacon Camp Rd.	49	35.1	49.0	13.9	1	CMS-2S	variable	1/8	3/16	130	Minor	Minor	Major	Fair	Maintenance due to delamination.
1985	Bacon Camp Rd.-M.P. 57	49	49.0	57.0	8.0	1	CMS-2S	variable	3/16	1/8	--	--	Minor	Major	Fair	
1985	M.P. 57-Hogback Summit	49	57.0	65.8	8.8	1	CMS-2S	variable	3/16	3/16	100	--	Minor	Major	Poor	
1985	O'Neil-Prineville*	370	9.5	17.7	8.2	1	CMS-2S	O-97	3/16	1/8	70-30	Minor	Minor	Minor	Poor	Grader laid.
1986	Pilot Butte-Powell Butte Jct.	7	1.1	4.3	3.2	2	CMS-2S	B	1/16	1/16	--	--	--	--	Good	Overlaid w/1-1/2 in. AC. Condition before overlay
1986	Horse Ridge-Fort Rock Rd.	7	18.1	21.0	2.9	3	CMS-2S	B	1/8	3/16	20	Minor	Minor	--	Fair-Good	
1986	G.I. Ranch-Harney Co. Line	7	75.0	84.0	9.0	1-1/2	CMS-2S	C	3/16	3/16	30-120	Minor	--	Minor	Fair-Good	US 20 test section
1986	Dairy-Wild Horse Cr.	20	19.0	25.0	6.0	1-1/2	CMS-2S	C	5/16	1/8	--	Heavy	--	Major	Poor	Maintenance due to base failure. Polymer seal.
1986	Sprague R. Rd.-Sycan Marsh Rd.	20	35.9	42.2	6.3	1-1/2	CMS-2S	C	1/8	3/16	60	Minor	--	Minor	Fair	Polymer seal.
1986	Sycan Marsh-Bly	20	42.2	54.0	11.8	1-1/2	CMS-2S	C	1/8	3/16	--	Minor	--	Minor	Fair	
1986	Powell Butte-Houston Lake Rd.	41	6.8	16.5	9.7	1-1/2	CMS-2S HFE-150	variable	3/16	1/4	100	--	Minor	Major	Poor	No seal initially. Pavement flushed when sealed. Overlaid w/OGEM in 1989.
1986	Ochoco Dam-Ranger Station	41	24.9	35.5	10.6	1-1/4	CMS-2S	oil mat	1/16	3/16	--	Minor	--	Minor	Fair	Overlaid w/OGEM in 1989.
1986	Keys Cr. Summit-Whiskey Cr.	41	73.4	81.6	8.2	2	CMS-2S	C?	1/16	1/16	--	--	--	--	Good	Overlaid w/OGEM in 1989.
1986	M.P. 89.6-Jct. John Day Hwy	41	89.6	98.4	8.8	1-1/2	CMS-2S	oil mat	3/16	1/16	--	Minor	--	Minor	Good	Chip sealed in 1986.
1986	M.P. 79.2-Bridge Cr.	53	79.2	86.9	7.7	1-1/2	CMS-2S	B	3/16	1/8	occasional	Minor	--	--	Fair	Early polymer seal. Recycled w/too much emulsion.
1986	Bridge Cr.-County Line	53	86.9	96.5	9.6	1-1/2	CMS-2S	C	1/8	3/16	60	Minor	--	Minor	Poor	Early polymer seal. Recycled w/too much emulsion.
1986	Lake Shore Dr.-Gr. Spgs. Hwy	270	62.4	68.8	6.4	1-1/2	CMS-2S	B	3/16	1/8	--	Minor	Minor	--	Good	
1986	Tub Springs Rd.-Antelope*	293	9.0	13.5	4.5	1 - 1-1/2	CMS-2S	oil mat	1/16	1/8	--	Minor	--	--	Good	
1986	McKay Cr.-Prineville*	360	23.7	26.3	2.6	1-1/2	CMS-2S	B-oil mat	1/8	1/4	--	Minor	Minor		Fair-Good	Partially (1 mi.) overlaid in 1986.
1986	Jct. Ochoco Hwy-Desch. Co. Line	371	0.0	7.6	7.6	3/4 - 1-1/2	HFE-150 CMS-2S	C & B & oil mat	1/4	3/16	--	Heavy	Minor	Minor	Fair-Good	
1986	Desch. Co. Line-Jct. Cent. Ore.	371	7.6	18.0	10.4	3/4 - 1-1/2	CMS-2S	oil mat	1/8	3/16	30-80	--	Minor	Minor	Fair-Good	
1987	Fort Rock Rd.-Crooked R. Hwy*	7	21.0	30.2	9.2	2	several	B	N/A	1/8	60-90	--	Minor	--	Poor-Good	East lane only. Overlaid w/OGEM in 1989.
1987	Whiskey Cr.-M.P. 89.6*	41	81.6	89.6	8.0	1 - 2-1/2	CMS-2S	variable	1/8	1/8	--	Heavy	--	Minor	Good	
1987	Jct. Hwy 97-Tub Springs Rd.*	293	0.0	9.0	9.0	1-1/4 - 2	CMS-2S EB HFE-150 WB	oil mat	1/16	1/8	--	Minor	--	--	Good	
1987	Jct. Hwy 97-Rammes Rd.*	360	0.0	9.0	9.0	1-3/4 - 2	CMS-2S NB HFE-150 SB	oil mat	3/16	1/8	--	Minor	Minor	--	Fair	No base. Recycled w/too much emulsion.
1987	Kiwa Springs-Sand Shed*	372	11.0	16.6	5.6	2	HFE 150	B	1/6	3/16	60-100	--	Minor	--	Good	
1987	Conant Basin Rd.-Shotgun Rd.*	380	20.7	29.8	9.1	1-1/2 - 2	CMS-2S	variable	1/8	1/8	--	Minor	--	--	Good	Overlaid w/O-9 in 1987.

TABLE 3 *(continued on next page)*

TABLE 3 *(continued)*

Year Built	Section (Note 1)	Hwy No.	M.P.	M.P.	Length (mi.)	Depth of CIR (in.)	Emulsion Used	Original Pavement (Note 2)	Rut Depth (in.) Lt.	Rut Depth (in.) Rt.	Thermal Crack Spacing (ft)	Flushing	Fatigue Cracks	Maint. Work	Rating	Notes
1988	Shaniko Jct.-Quaale Rd.	4	67.2	75.6	8.4	1-1/2	HFE-150	B	1/8	3/16	--	--	--	--	Poor-Fair	M.P. 67.2-69.2 (SB) failed due to high asphalt - 2" ruts.
1988	Shaniko Jct.-Quaale Rd.	4	75.6	78.6	3.0	1-1/2	HFE-150	0-11	1/16	1/8	--	--	Minor	--	Good	
1988	Shaniko Jct.-Quaale Rd.	4	78.6	80.0	1.4	1-1/2	HFE-150	0-11	0	0	--	--	--	--	Good	
1988	M.P. 152-Cal. Line*	19	152.0	157.7	5.7	2	CMS-2S	B?	3/16	3/16	--	Minor	Minor	Major	Poor	Unstable
1988	Houston Lake Rd.-Prineville*	41	16.5	18.0	1.5	2	HFE-150 HFE-150S	B&C	1/16	1/16	--	--	--	--	Good	Scheduled to be overlaid w/AC
1988	Prineville-Ochoco Dam*	41	19.4	24.9	5.5	2	HFE-150	B & C & oil mat	1/16	1/8	--	Minor	--	Minor	Fair-Good	Overlaid w/OGEM in 1989
1988	Ochoco Ranger St.-Rush Cr.*	41	35.5	45.0	9.5	2	CMS-2S HFE-150	variable	0	1/16	--	--	--	--	Good	
1988	Wheeler Co. Line-W. Brand Cr.*	41	50.1	60.3	10.2	2	HFE-150	variable	1/8	0	--	--	--	--	Good	
1988	Lake Abert-Valley Falls*	49	87.0	89.9	2.9	1 - 1-1/2	CMS-2S	oil mat	3/16	1/4	--	Minor	Minor	Major	Poor	Fat oil mat.
1988	Merrill-Jct. Hatfield Hwy*	50	13.7	16.3	2.6	2	CMS-2S	B	1/8	1/16	100-200	Minor	--	--	Good	
1988	Jct. Ochoco Hwy-Burma Rd.*	380	0.0	11.8	11.8	2	HFE-150	C?	1/8	1/16	--		Minor	--	Fair	No seal - raveled.
1988	Burma Rd.-Conant Basin Rd.*	380	11.8	20.7	8.9	2	HFE-150	C?	1/16	1/16	--		--	--	Good	Sealed in 1989.
1988	Malin HWY-Calif. Line*	426	0.0	2.4	2.4	2	CMS-2S	Mod-B w/seal	1/8	1/8	--		Minor	--	Good	Stripped.
1988	Crater Lake Hwy (Ft. Klamath-Crooked Cr.)	22	90.07	95.4	5.3	2	HFE-100S	E							Good	
1988	12 mi South of Clackamas Ranger Station	FS 46	-	-	3.2	2	CMS-2S	B	<1/4	<1/4	--	--	--	--	Very Good	

Notes:

1. Sections with an asterisk (*) were recycled by state forces.
2. B, C, and E denote the class of aggregate gradation as specified in Oregon's standard specifications (reference 6). 0-9 denotes an oil mat having a thickness of 9/100-in. while 0-11 denotes an oil mat with a thickness of 11/100-in.

TABLE 4 BEFORE AND AFTER RIDE DATA FOR SELECTED CIR PROJECTS

Project	Year Constructed	Average Ride (in./mi)		Score Rating	
		Before	After	Before	After
Harney Co. Line-Hogback Summit	1985	175	62	Rough	Smooth
Warm Springs	1986	69	69	Smooth	Smooth
Powell Butte-Prineville	1986	162	112	Rough	Slightly Rough
Lake of the Woods	1986	69	61	Smooth	Smooth

ing time and densification caused by traffic. (Bulk gravities increase with time as indicated in Table 5.) In all cases in which fatigue was monitored over time, the fatigue lives increased significantly. This increase may also be attributed to the additional cure time and densification caused by traffic. The fatigue lives are comparable to those of conventional mixes (i.e., 10,000 to 50,000 repetitions to failure).

In addition to modulus and fatigue, Marshall stability and flow were monitored over time for the 1986 projects. In all cases, the stabilities increased but the flow values remained slightly high. These results generally reflect, and thus support, the modulus and fatigue test results.

Life Expectancy

On the basis of present data for low-volume roads, the following life expectancies may be warranted:

1. 2-in. AC—10 to 15 years,
2. CIR (with chip seal)—7 to 8 years, and
3. CIR (with OGEM)—7 to 12 years.

These life expectancies are based on discussions with Oregon Department of Transportation (ODOT) personnel. The CIR (with chip seal) values are based on a study of 32 projects constructed between 1984 and 1988, which are currently serving as wearing courses on low-volume roads (*8*). Fourteen of the projects had experienced no significant patching after an average of 4.5 years. Total service lives of 8 years were predicted for these projects. Eighteen projects had experienced patching after an average of 2 years but were still predicting total service life of 7 years.

ECONOMICS OF CIR

Typical Costs

Typical costs for construction and maintenance of CIR versus hot-mix projects are presented in Table 6. Construction and maintenance costs are based on conversations with ODOT personnel and reflect typical values in the state of Oregon.

TABLE 5 SUMMARY OF MIX PROPERTY TEST RESULTS

Project	Test Period (Months After Construction)	Average Bulk Specific Gravity	Average Resilient Modulus* (ksi)	Average Fatigue Life**	Average Marshall Stability*** (lb)	Average Flow (in./100)
Century Drive	15	-	230	-	-	-
	17	2.203	322	77800	-	-
	63	2.273	713	138184	2410	17
Harney Co. Line - Hogback Summit	3	-	293	-	-	-
	5	1.946	403	35072	-	-
	39	2.030	508	108865	788	33
	51	2.005	485	175000+	1607	19
Drews Gap-Lakeview	3	1.940	278	3424	-	-
	5	2.005	323	19317	-	-
	39	2.116	499	61805	1196	22
	51	2.152	531	48076	2049	20
Warm Springs	3	2.160	305	11030	694	59
	12	2.333	242	50010	861	20
	24	2.273	377	53965	1106	21
	36	2.381	526	150000+	1181	18
Lake of the Woods	3	2.059	513	5860	605	29
	12	2.092	504	34261	614	20
	24	2.132	530	78731	1171	24
	36	2.141	727	250000+	1597	17

* ASTM D4123 - Tests run at 23°C, 100 microstrain, and at a load frequency of 1 hertz
** After reference 7
*** ASTM D1559

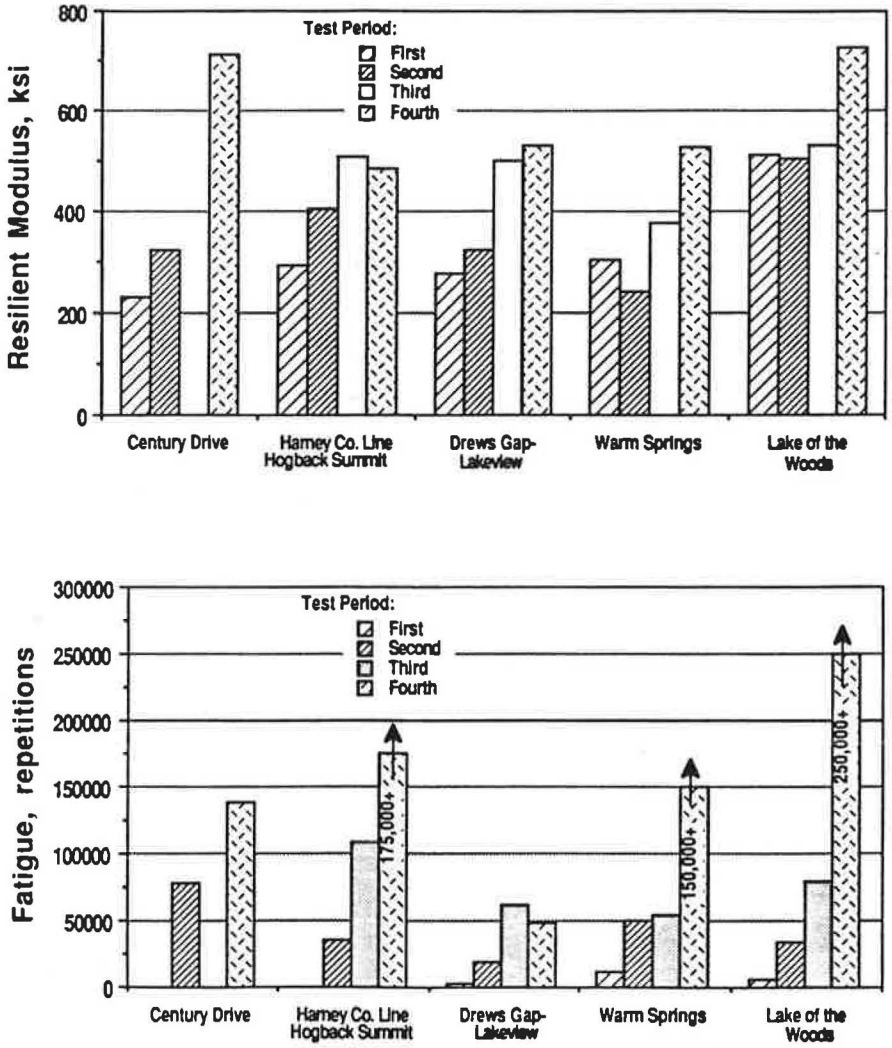

FIGURE 5 Resilient modulus (*top*) and fatigue (*bottom*) test results.

Life Cycle Cost Analysis

Life cycle costs were analyzed for two CIR alternates and a hot-mix overlay alternate. Equivalent annual cost analysis was chosen to simplify the comparison between alternates of differing economic lives. It is assumed that when an alternate's economic life is reached, the same treatment will be repeated, essentially in perpetuity. For example, every 7 or 8 years, the basic CIR alternate is recycled and given a chip seal. Every 10 to 15 years, the 2-in. hot-mix overlay is repeated.

Variables in the life cycle analysis include:

1. Interest rate, 8 percent;
2. Construction costs, see Table 6;
3. Maintenance costs, see Table 6; and
4. Life expectancy (CIR—7 to 8 years, CIR with OGEM—7 to 12 years, and hot mix—10 to 15 years).

The results of the analysis are presented in Table 7. In general, the results indicate the following:

1. Basic CIR represents the lowest first cost of the alternates considered. This means that more miles can be preserved on limited budgets. This, in turn, means it is more likely that more miles of roads can be saved before they deteriorate to the point where expensive reconstruction becomes necessary.
2. Life cycle costs for basic CIR are clearly the lowest of the alternates considered. This means that the easy choice for present budgets will also produce optimum results for future budgets.
3. Best- and worst-case comparisons show CIR with chip seal to have the lowest life cycle costs and the 2-in. hot-mix overlay to have highest life cycle cost, with CIR with OGEM in the middle. Ranges of costs do overlap.
4. CIR's advantages would increase in cases where haul distances are large and hot-mix suppliers are few or not competitive. CIR's advantages would decrease with short haul distance and highly competitive hot-mix suppliers.
5. Analysis does not include user costs. Inclusion of user costs would decrease the advantages of CIR.

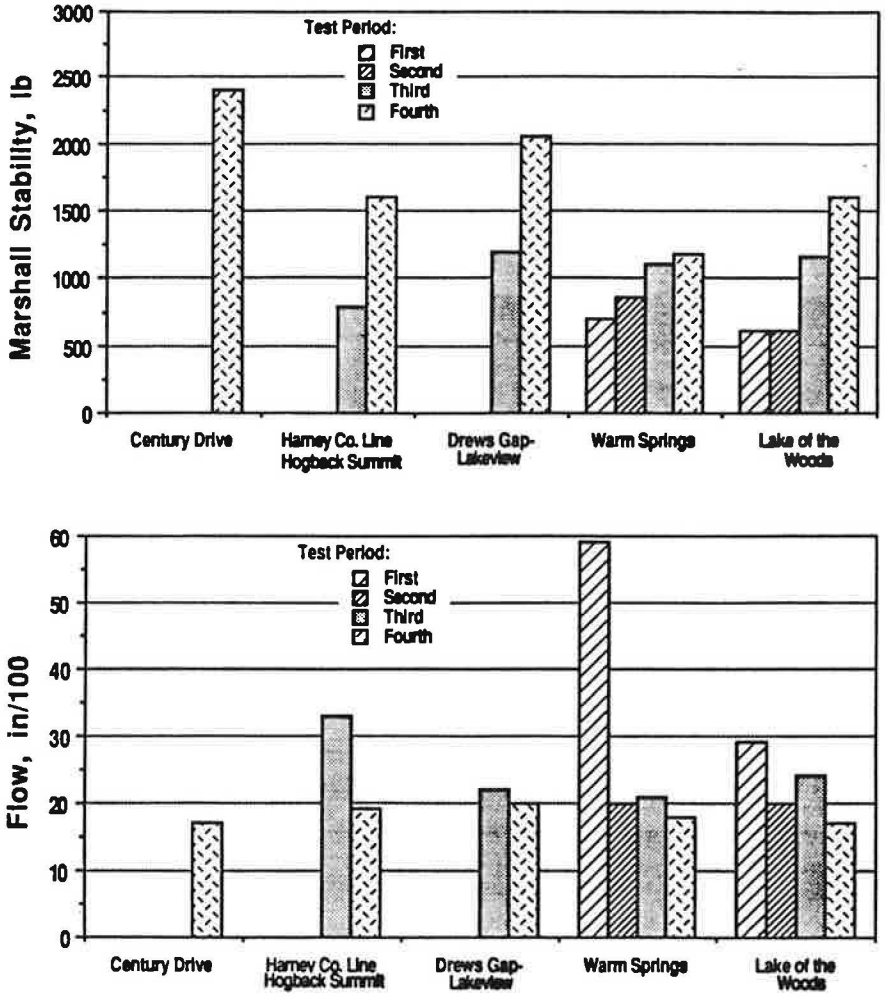

FIGURE 6 Marshall stability (*top*) and flow (*bottom*) test results.

TABLE 6 TYPICAL COSTS FOR CIR VERSUS HOT MIX—ODOT

Item	Construction Cost	Maintenance Cost
CIR (2-in.) with chip seal	$1.70 - $2.10/sq.yd.	Best Case: Maintenance of $1200/mi/yr beginning in 6th year and increasing 25% each year. Worst Case: Maintenance of $1200/mi/yr beginning in 3rd year and increasing 25% each year
CIR (2-in.) with 1½-in. OGEM	$2.20 - $3.00/sq.yd.	$8,700/mile after 7 years (chip seal)
2-in. Hot Mix Overlay	$2.75 - $4.00/sq.yd.	Maintenance of $1200/mi/yr beginning in 8th year and increasing 25% each year

TABLE 7 RESULTS OF LIFE CYCLE COST ANALYSIS

Item	Equivalent Annual Cost Per Mile
CIR (2-in.) with chip seal	$4,500 - $6,600
CIR (2-in.) with 1½-in. OGEM	$4,800 - $8,100
2-in. Hot Mix Overlay (with maintenance)	$5,400 - $8,600

6. The fact that the hot-mix overlay alternate has greater structural section (thickness) than the CIR alternates is not reflected in this economic analysis.

SUMMARY AND CONCLUSIONS

As previously stated, CIR is effective in restoring cracked, broken, raveled, or rough pavements where ADT is 5,000 or less. CIR may also be used for leveling and as a base for overlay. The volume limitation of ADT of 5,000 or less is a result of traffic control problems during construction rather than being related to load. Because of this, CIR is now being considered for higher-volume highways (multilane, including Interstates) where traffic can be effectively controlled. Studies are underway to determine if CIR may be used on pavement with stripping problems. CIR should not be used where pavement has rutted because of a fat mix, or where conditions are too cold and damp to allow adequate curing of the emulsion.

When considering the use of CIR, potential problems must be considered. Implications for traffic control must be thoroughly evaluated. If CIR is to be followed with a chip seal, the usual precautions regarding loose chips must be taken. It is essential that climate conditions allow for the curing of the asphalt emulsion; several warm days and nights are required to allow for adequate curing.

When CIR can be used, it represents a cost-effective method for restoring asphalt pavements of low-volume roads. Ride quality is superior to any type of patching. As has been demonstrated by life cycle cost analysis, CIR is more cost-effective than the application of hot-mix overlays. From a first-cost standpoint, only simple fog seals or chip seals are more economical, and these do nothing to level the pavement.

The preceding discussion leads to the following conclusions:

1. Current state-of-the-art in recycling equipment allows effective cold recycling of asphalt pavements.
2. CIR projects are performing well in the high desert environment of central and eastern Oregon. Performance evaluations in western Oregon are just underway.
3. CIR may be the most cost-effective restoration treatment for low-volume asphalt pavements. Life cycle cost analysis indicates a clear preference for CIR over conventional hot-mix overlays.
4. CIR provides environmental and energy conservation benefits realized through savings in materials and fuel.

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Design of Large-Stone Asphalt Mixes for Low-Volume Roads Using 6-in.-Diameter Marshall Specimens

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Premature rutting of road pavements constructed for hauling coal and logs is common. Although these roads carry low volumes of traffic, they are subjected to heavy and channelized wheel loads. Unfortunately, conventional asphalt mixes containing aggregates less than 1-in. maximum size in the base or binder course tend to develop premature rutting under these conditions. Many asphalt technologists believe that the use of large-size stone (maximum size of more than 1 in.) will minimize or eliminate this problem. Large-stone mixes are also economical for low-volume roads because of substantially reduced asphalt contents. However, most agencies use the Marshall design procedure (ASTM D1559), which uses a 4-in.-diameter compaction mold intended for mixes containing aggregate up to 1-in. maximum size only. This has inhibited the use of large-stone mixes. A standard method for preparing and testing 6-in.-diameter specimens has been presented. Mixes containing aggregate up to 2-in. maximum nominal size can be tested. A typical mix design using 6-in. specimens for a coal-haul road in Kentucky is given. Construction data and experience gained from field projects in Kentucky are also included. The proposed test method may be useful in determining the optimum asphalt content of large-stone asphalt mixes that are recommended for use on low-volume roads subjected to heavy and channelized wheel loads.

Premature rutting of road pavements constructed for hauling coal and logs is quite common. The problem of these roads that provide the essential first link in the transportation chain that brings the products of mines and forest to market is unique. Although these roads carry low volumes of traffic, they are subjected to heavy and channelized wheel loads. Coal-haul roads in Kentucky have been reported to carry trucks with gross loads ranging from 90,000 to 150,000 lb. Tire pressures are also higher than generally encountered, ranging from 100 to 130 pounds per square inch (psi).

Unfortunately, conventional hot-mix asphalt (HMA) mixes containing aggregates less than 1-in. maximum size tend to develop premature rutting under these conditions. Many asphalt technologists believe that the use of large-size stone (maximum size of more than 1 in.) will minimize or eliminate this problem. Large-stone mixes are also economical for low-volume roads because of substantially reduced asphalt contents. A thin asphalt surfacing needs to be provided over the large-stone asphalt mix to obtain smooth surface.

Marshall mix design procedures are used by 76 percent of the states in the United States according to a survey conducted in 1984 (*1*). The equipment specified in the Marshall procedure (ASTM D1559) consists of a 4-in.-diameter compaction mold that is intended for mixtures containing aggregate up to 1-in. maximum size only. This has also inhibited the use of HMA containing aggregate larger than 1 in. because it cannot be tested by the standard Marshall mix design procedures. There are other test procedures such as gyratory compaction, TRRL (Transport and Road Research Laboratory, U.K.) refusal test, and Minnesota DOT vibrating hammer, which use 6-in.-diameter molds accommodating 1½- to 2-in. maximum aggregate size (*2*). However, most agencies are reluctant to buy new equipment because of cost and complexity. They tend to prefer and use the existing equipment and methodology (such as Marshall test) with some modifications.

The term "large-stone" is a relative one. For the purpose of this report, large-stone mix is defined as an aggregate with a maximum size of more than 1 in., which cannot be used in preparing standard 4-in.-diameter Marshall specimens.

BACKGROUND OF DEVELOPMENT

Pennsylvania Department of Transportation (PennDOT) implemented Marshall mix design procedures in the early 1960s. The Marshall method was generally based on ASTM D1559 (*Standard Test Method for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus*). ASTM D1559 specifies the use of 4-in.-diameter specimen mold for mixes containing aggregate up to 1-in. maximum size. The compaction hammer weighs 10 lb and a free fall of 18 in. is used. It became apparent that ASTM D1559 could not be used for designing Pennsylvania ID-2 binder course mix and base course mix, which specified maximum permissible sizes of 1½ and 2 in., respectively. Therefore, PennDOT completed a study in 1969 to develop the equipment and procedure for testing 6-in.-diameter specimens (*3*).

A series of compaction tests was run using 4- and 6-in.-diameter specimens of wearing and binder mixes. The nominal height of the 6-in.-diameter specimen was increased to 3¾ in. to provide the same ratio of diameter to height that is used for a 4-in.-diameter × 2½-in.-high specimen. When the 6-in. compactor was designed, it was assumed that the weight of the hammer should be increased in proportion to the face area of the Marshall specimen, and the height of hammer drop and the number of blows on the face of the specimen should remain the same as that used for the 4-in.-diameter specimens. The weight of the hammer, therefore, was increased from 10 to 22.5 lb, and the hammer drop was maintained at

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18 in. with 50 blows on each face. However, the initial test data indicated that the energy input to the specimen during compaction should have been based on ft-lb/in.^{3} of specimen volume instead of ft-lb/in.^{2} of the specimen face. Therefore, to obtain the same amount of energy input per unit volume in a 6- by 3¾-in. specimen, the number of blows had to be increased from 50 to 75. The comparative compaction data presented in Table 1 substantiates this result. On the basis of these data, it was specified that a 6-in.-diameter, 3¾-in.-high specimen should be compacted with a 22.5-lb hammer, free fall of 18 in., and 75 blows per face. The details of equipment, such as mold, hammer, and breaking head are given in Pennsylvania Test Method 705 developed by Kandhal and Wenger (*4*).

Preliminary test data obtained in 1969 during the developmental stage are given in Tables 2 and 3 for ID–2 wearing course (maximum aggregate size ½ in.) and ID–2 binder course (maximum aggregate size 1½ in.) mixtures, respectively. The data indicate that reasonably close compaction levels are achieved in 4- and 6-in.-diameter molds when the number of blows for 6-in. specimens is 1½ times that used for 4-in. specimens. Marshall void parameters such as percent air voids, percent VMA, and percent VFA are also reasonably close. The stability and flow values will increase when a larger 6-in. specimen is tested in lieu of a 4-in. specimen of the same mix. Table 3 indicates that a preliminary stability ratio (stability of 6-in. specimen/stability of 4-in. specimen) of 2.12, and a flow ratio (flow of 6-in. specimen/flow of 4-in. specimen) of 1.62 were obtained for the binder course mix. Additional comparative test data (4- versus 6-in.-diameter specimens) obtained by various agencies will be discussed later in this report.

The next step taken by PennDOT in 1970 was to evaluate the repeatability of the test results using 6-in. equipment. A binder course mix was used to compact nine 4-in.-diameter specimens and ten 6-in.-diameter specimens. Statistical analysis of stability, flows, and air voids data presented in Tables 4 and 5 indicates better repeatability of 6-in. specimens compared to 4-in. specimens when testing a large-stone mix. This repeatability is evident from significantly lower values of the coefficient of variation obtained on 6-in. specimens. It is also expected because of the decreased ratio of aggregate maximum size to specimen diameter. The coefficients of variation of stability and flow were reduced by at least 50 percent.

ASTM Subcommittee D04.20 on Mechanical Tests of Bituminous Mixes appointed a task force in December 1988 to develop an ASTM standard test for preparing and testing 6-in.-diameter Marshall specimens. The chairman of this task force prepared a draft for this proposed standard that was published by Kandhal (*5*). The proposed standard follows ASTM D1559–82 (*6*), which is intended for 4-in.-diameter specimens, except with significant differences in the following areas:

1. Equipment for compacting and testing 6-in.-diameter specimens, such as molds and breaking head (Section 3);
2. Because the hammer weighs 22.5 lb, only a mechanically operated hammer is specified (Section 3.3);
3. About 4050 g of mix is required to prepare one 6-in. Marshall specimen, compared with about 1200 g for a 4-in. specimen;
4. The mix is placed in the mold in two approximately equal increments, and spading is specified after each increment (Section 4.5.1). Past experience has indicated that spading is necessary to avoid honeycombing on the outside surface of the specimen and to obtain the desired density. Mixing and compaction temperatures remain the same as for 4-in.-diameter specimens.
5. The number of blows needed for 6-in.-diameter and 3¾-in.-high specimen is 1½ times the number of blows needed

TABLE 1 COMPARATIVE DATA FOR 4- VERSUS 6-in.-DIAMETER SPECIMENS, 1969 DATA

	WEARING MIX				BINDER MIX		
Specimen Diameter, in.	4	6	6	6	4	6	6
Specimen Height, in.	2.50	3.75	2.50	3.75	2.50	3.75	3.75
Hammer Weight, lbs.	10	22.5	22.5	22.5	10	22.5	22.5
Hammer Drop, in.	18	18	18	18	18	18	18
No. of Blows/Face	50	50	50	75	50	50	75
Energy Input :							
Ft.lb/sq. in. of Specimen Face	119.4	119.4	119.4	179.1	119.4	119.4	179.1
Ft.lb/cu. in. of Specimen	47.7	31.8	47.7	47.7	47.7	31.8	47.7
Percent Compaction of Theor. Max. Specific Gravity	94.2	92.9	93.9	94.0	97.5	96.4	97.4
Percent Void Content	5.8	7.1	6.1	6.0	2.5	3.6	2.6
Stability, lbs.	2049	5316	-	-	1622	3785	3440
Flow, Units	10.0	20.4	-	-	10.8	20.8	17.5

TABLE 2 COMPARATIVE TEST DATA FOR 4- VERSUS 6-in.-DIAMETER SPECIMENS (WEARING COURSE)

Source : Pennsylvania Dept. of Transportation (1969 Data) Mix type : ID - 2 Wearing Course.
Aggregates : Limestone coarse aggregate and limestone fine aggregate.
Design Gradation (% Passing) :

2"	1-1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
-	-	-	-	100	95	63	43	28	18	12	8	4.5

	4" Specimen	6" Specimen		4" Specimen	6" Specimen
No. of Blows	50	75	Stability, pounds	2049	-
% Compaction	94.2	94.0	Flow, units	10.0	-
% Air Voids	5.8	6.0			
% VMA	18.8	18.9	Remarks : Data on Stability and Flow of 6" specimens is not available.		
% VFA	69.4	68.4			

Remarks: Results are based on average of six 4"-dia. specimens and three 6"-dia. specimens.

for 4-in.-diameter and 2½-in.-high specimen to obtain equivalent compaction level (Note 4).

6. Stability correlations ratios have been revised and are presented in Table 6. These ratios are based on percentage of increase or decrease in specimen volumes, similar to ASTM D1559.

Relative sizes of mold and hammer assembly for compacting 4- and 6-in. specimens are shown in Figure 1. The same mechanical compactor can be used for compacting both types of specimens (Figure 2). Figures 3 through 6 show the details of the test equipment.

Because the hammer weighs 22.5 lb and the number of blows on each side is 75 or 112 depending on the anticipated traffic, some crushing of the aggregate at the surface has been observed. However, its effect on Marshall properties is believed to be minimal.

Vigorous spading in the mold is necessary to prevent voids near the large stones. The mix should not be allowed to cool below the intended compaction temperature.

TABLE 3 COMPARATIVE TEST DATA FOR 4- VERSUS 6-in.-DIAMETER SPECIMENS (BINDER COURSE)

Source : Pennsylvania Dept. of Transportation (1969 Data) Mix type : ID - 2 Binder Course.
Aggregates : Limestone coarse aggregate and limestone fine aggregate.
Design Gradation (% Passing) :

2"	1-1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
100	100	95	-	58	-	34	25	20	15	10	7	3

	4" Specimen	6" Specimen		4" Specimen	6" Specimen
No. of Blows	50	75	Stability, pounds	1622	3440
% Compaction	97.5	97.4	Flow, units	10.8	17.5
% Air Voids	2.5	2.6			
% VMA	14.7	15.1	Stability Ratio	2.12	
% VFA	83.2	83.0	Flow Ratio	1.62	

Remarks : Results are based on average of 3 specimens each.
Stability Ratio = Stability of 6" specimen / Stability of 4" specimen.
Flow Ratio = Flow of 6" specimen / Flow of 4" specimen.

TABLE 4 REPEATABILITY OF MARSHALL TEST FOR 4-in.-DIAMETER SPECIMENS (BINDER COURSE MIX), 1970 DATA .

	Stability Pounds	Flow 0.01 Inch	Voids Percent
	1290	9.0	3.2
	1750	13.5	3.4
	1635	17.0	2.8
	2035	10.0	3.0
	1540	22.0	3.2
	2090	13.5	2.8
	1975	19.0	2.3
	2200	14.0	2.6
	1620	11.5	2.6
N	9.0	9.0	9.0
Mean	1793	14.4	2.9
Std Dev	300	4.2	0.4
Coeff of Var.(%)	16.7	29.2	13.8

TABLE 5 REPEATABILITY OF MARSHALL TEST FOR 6-in.-DIAMETER SPECIMENS (BINDER COURSE MIX), 1970 DATA

	Stability Pounds	Flow 0.01 Inch	Voids Percent
	4850	13.0	3.2
	4653	18.0	3.0
	4605	19.0	2.5
	5428	15.0	2.7
	5188	15.0	2.7
	4960	15.5	2.7
	5232	18.0	2.7
	5886	19.0	2.4
	-	-	2.8
	-	-	2.2
N	8	8	10
Mean	5100	16.6	2.7
Std Dev	427	2.2	0.3
Coeff of Var.(%)	8.4	13.2	11.1

Note : Stability ratio and flow ratio (6" versus 4" diameter) in these repeatability experiments were determined to be 2.81 and 1.15, respectively.

TABLE 6 STABILITY CORRELATIONS RATIOS[A]

Approximate Thickness of Specimen[B]		Volume of Specimen, cm^3	Correlation Ratio
in.	mm		
3-1/2	88.9	1608 to 1626	1.12
3-9/16	90.5	1637 to 1665	1.09
3-5/8	92.1	1666 to 1694	1.06
3-11/16	93.7	1695 to 1723	1.03
3-3/4	95.2	1724 to 1752	1.00
3-13/16	96.8	1753 to 1781	0.97
3-7/8	98.4	1782 to 1810	0.95
3-15/16	100.0	1811 to 1839	0.92
4	101.6	1840 to 1868	0.90

[A] The measured stability of a specimen multiplied by the ratio for the thickness of the specimen equals the corrected stability for a 3-3/4-in. (95.2 mm) thick specimen.

[B] Volume - thickness relationship is based on a specimen diameter of 6 in. (152.4 mm).

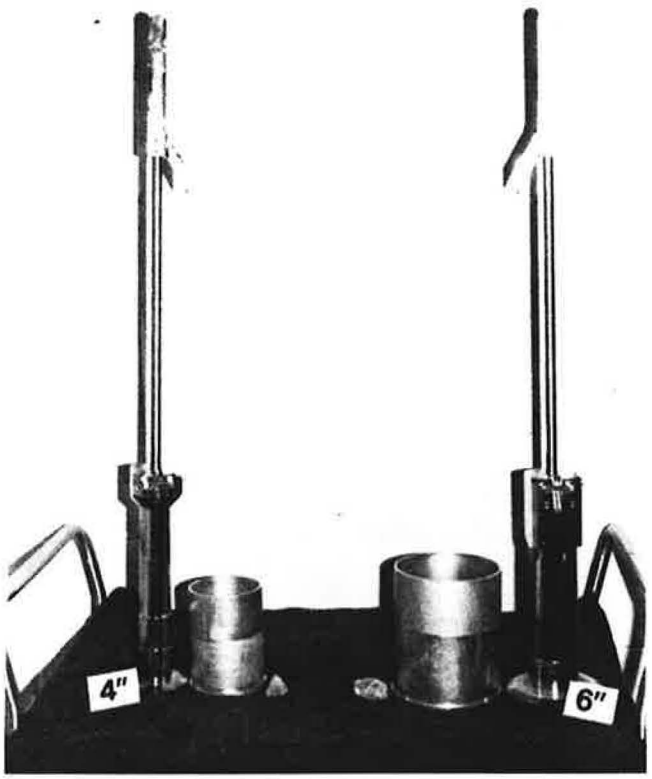

FIGURE 1 Mold and hammer assembly for 4- and 6-in.-diameter specimens. Aggregate particles of 1- and 2-in. maximum size are also shown.

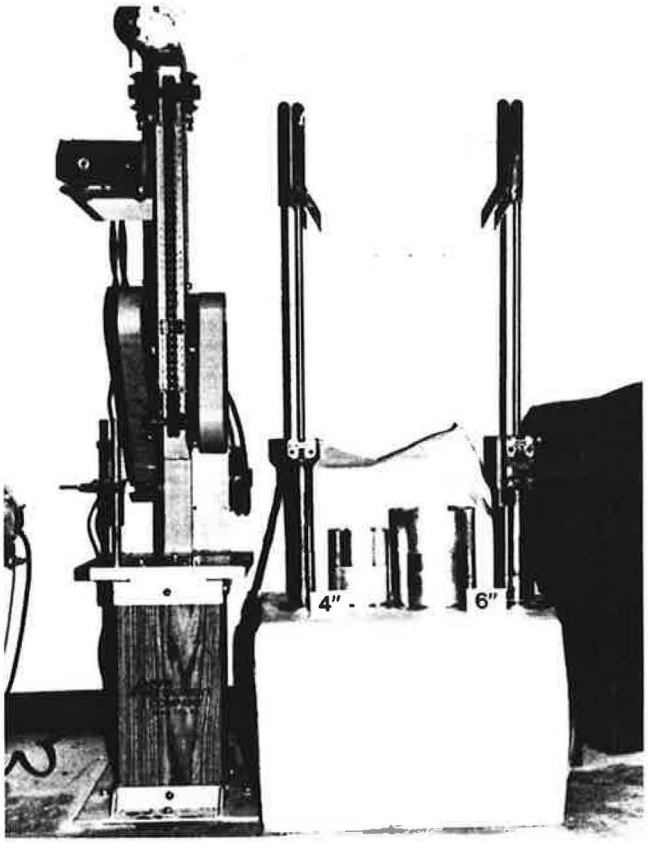

FIGURE 2 Compaction equipment for 4- and 6-in.-diameter specimens.

At the present time, there are two known suppliers of 6-in. Marshall testing equipment in the U.S.:

1. Pine Instrument Company
   101 Industrial Drive
   Grove City, PA 16127

2. Rainhart Company
   P.O. Box 4533
   Austin, TX 78765

If a mechanical compactor is already on hand, the following additional equipment (estimated cost $1,800) must be purchased:

1. 6-in. complete mold assembly consisting of compaction mold, base plate, and collar (three are recommended);
2. 6-in. additional compaction molds (six are recommended);
3. 6-in. compaction hammer (two are recommended);
4. 6-in. mold holder (ensure that the spring is strong);
5. 6-in. breaking head assembly;
6. Specimen extractor for 6-in. specimen; and
7. 6-in. paper discs (box of 500).

4- VERSUS 6-in.-DIAMETER SPECIMENS

After the preliminary developmental work done by PennDOT during 1969 and 1970, there was minimal use of 6-in. Marshall equipment until 1987. Interest in this equipment was revived because various agencies and producers wanted to test large-stone mixes for minimizing or eliminating rutting of HMA pavements as discussed earlier. These agencies (including PennDOT) and producers who procured the 6-in. Marshall testing equipment ran a limited number of tests to verify the degree of compaction obtained in 6-in. mold compared to 4-in. mold. Also, a need was felt to verify the stability ratio (stability of 6-in. specimen/stability of 4-in. specimen) and the flow ratio (flow of 6-in. specimen/flow of 4-in. specimen) obtained in PennDOT's preliminary work. These tests were necessary so that minimum stability values, and the range of flow for 6-in. specimens could be derived from the values specified for 4-in. specimens. Personal contacts were made with various agencies and producers to obtain comparative test data.

Table 7 presents the stability and flow ratio values obtained by two agencies and two producers (Jamestown Macadam, New York, and American Asphalt Paving Co., Pennsylvania) on large-stone base or binder mixes (maximum aggregate size 1½ to 2 in.). The average of 11 stability ratios is 2.18, and the average of 11 flow ratios is 1.44. These values are close to theoretically derived values.

From a theoretical viewpoint, an external load applied to the circumference of a cylinder may be considered as acting directly on the diametrical cross section of the cylinder. This permits calculation of the stress in pounds per square inch. The standard 6-in. specimen is 3¾ in. high, which gives a

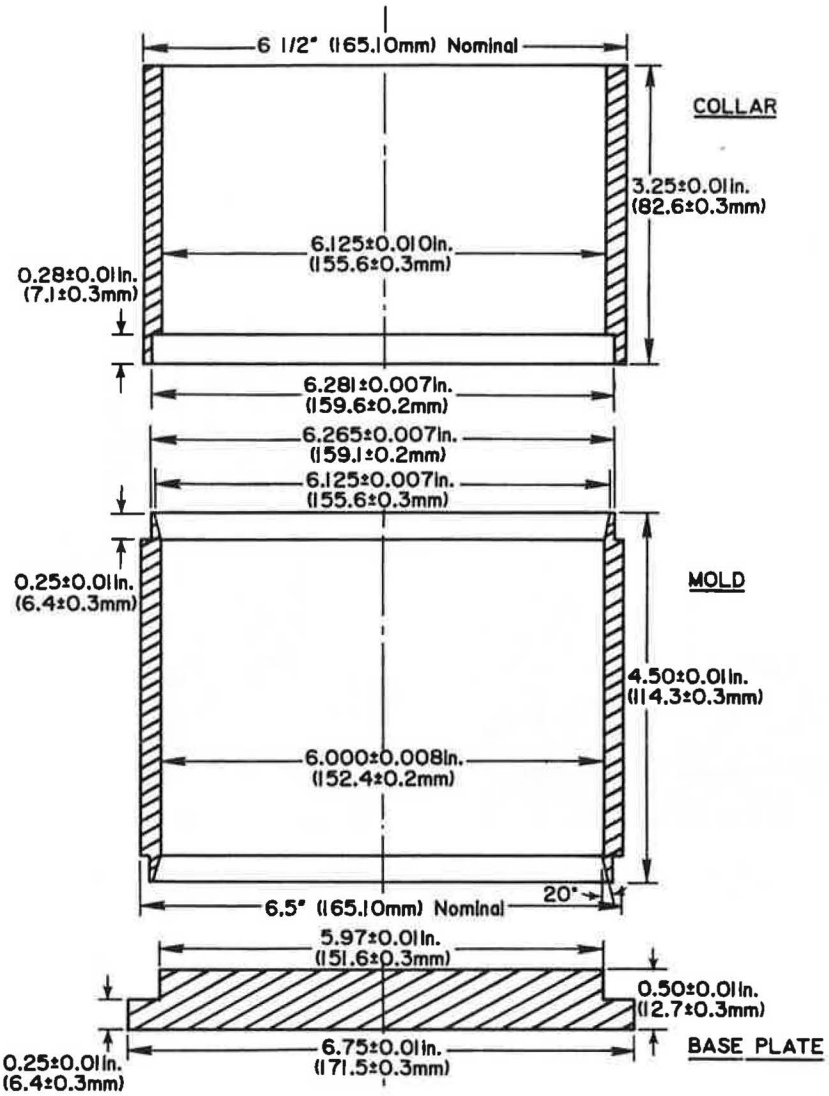

FIGURE 3 Compaction mold.

diametrical cross section of 22.5 in.^{2}. The standard 4-in. specimen is 2½ in. high and it has a diametrical cross section of 10.0 in.^{2}. Therefore, on the basis of unit stress, the total load on a 6-in. specimen should be 2.25 times the load applied to a 4-in. specimen of the same mix. This means the stability ratio should be 2.25.

Flow units measured by the testing machine are the values for the total movement of the breaking heads to the point of maximum stability. When flow is considered on a unit basis (inches per inch of diameter), the flow value for a 6-in. specimen will be 1.5 times that of a 4-in. diameter specimen. This means the flow rato should be 1.5.

Surprisingly, the average stability and flow ratio of specimens compacted with 75 and 112 blows (4- and 6-in. molds, respectively) are 2.28 and 1.49, which are close to the theoretically derived values of 2.25 and 1.50, respectively.

It is recommended that the minimum Marshall stability requirement for 6-in.-diameter specimens should be 2.25 times the requirement for 4-in.-diameter specimens. For example, if 1,000-lb minimum stability is currently being specified using ASTM D1559 (4-in. specimen), then 2,250-lb minimum stability should be specified for large-stone mixes using the 6-in. Marshall testing equipment.

Similarly, the range of flow values for 6-in. specimens should be adjusted to 1½ times the values required for 4-in. specimens. For example, if the specified range for 4-in. specimens is 8 to 18, it should be adjusted to 12 to 27 for 6-in. specimens.

TYPICAL MIX DESIGN USING 6-in. SPECIMENS

Kentucky DOH has completed a substantial number of large-stone mix designs for coal-haul roads using the 6-in. Marshall testing equipment. The contractor is required to buy the testing equipment for the project so that proper quality control is maintained. Kentucky DOH Class K base mix has been used on coal-haul roads carrying heavy trucks (gross loads varying from 90,000 to 150,000 lb or more), as mentioned

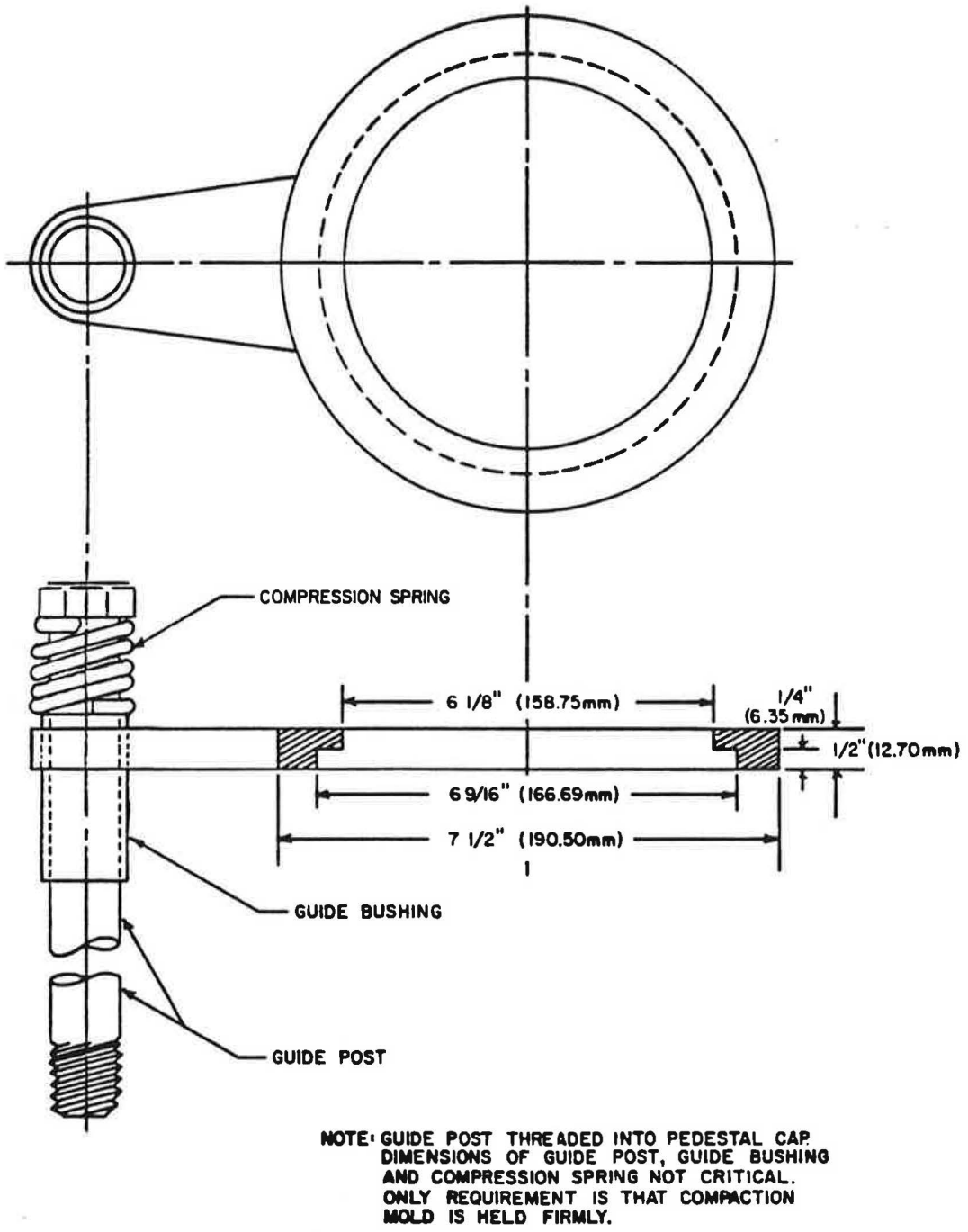

FIGURE 4 Specimen mold holder.

earlier. Tire pressures are also higher than generally encountered, ranging from 100 to 130 psi (*7*).

Table 8 presents the typical Marshall mix design data for one project along with the gradation used for Class K base. The mix contains limestone aggregates and a maximum aggregate size of 2 in. with a substantial amount of material retained on the 1-in. sieve. This results in substantial amount of 1- to ¾-in. material in the mix. The mix design was developed using a 6-in. mold and 112 blows on each side. Asphalt content was varied from 3.2 to 4.0 percent in 0.4 percent increments. Either AASHTO Gradation #467 (1½ in. to No. 4) or Gradation #4 (1½ to ¾ in.) is used for coarse aggregate to incorporate +1-in. material in the mix. The following preliminary design criteria have been used by Kentucky DOH on the basis of laboratory and field evaluation of such mixes:

Stability	3,000 lb, minimum
Flow	28, maximum
Air Voids	4.5 ± 1.0 percent
VMA	11.5 percent, minimum

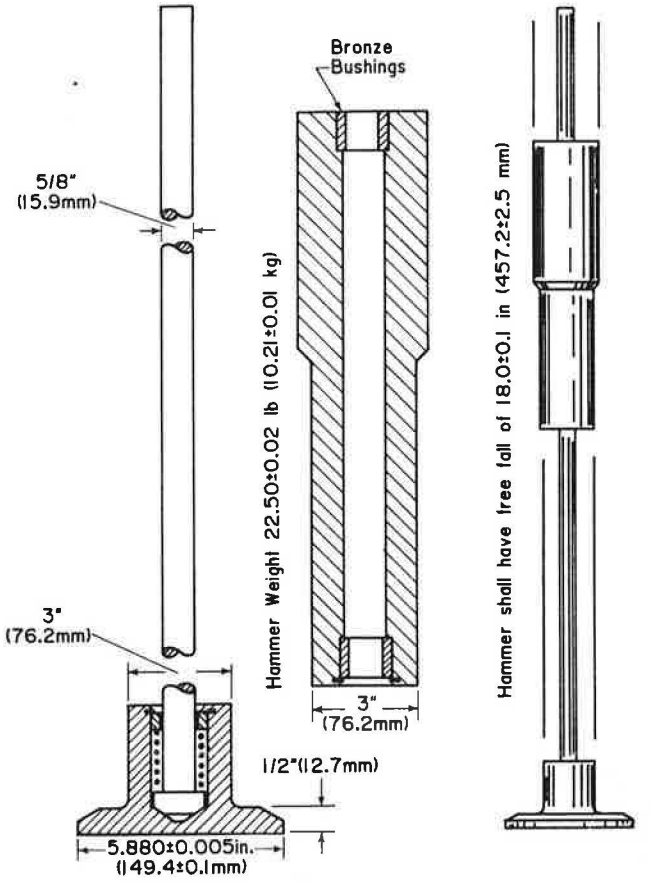

FIGURE 5 Compaction hammer.

FIELD TRIALS AND DATA

Kentucky DOH's experimental specifications require construction of a control strip (at least 500 ft long and 12 ft wide) at the beginning of construction of Class K base. Construction of the control strip is accomplished using the same compaction equipment and procedures to be used in the remainder of the Class K base course. After initial breakdown rolling and two complete coverages of the pneumatic-tired intermediate roller, three density measurements are made at randomly selected sites. Measurements are repeated at the same sites after each two subsequent complete coverages by the pneumatic-tired roller until no further increase in density is obtained. After the completion of the control strip, 10 field density measurements are performed at random locations. The target density to be used for the compaction of the remainder Class K base is the average of these 10 measurements. However, the target density obtained from the control strip should be no greater than 97.0 percent nor less than 93.0 percent of the measured maximum specific gravity (Rice specific gravity) as determined by AASHTO T209. The minimum acceptable densities for the remainder project are as follows:

Single test:	96.0 percent of the target density.
Moving average of last 10 tests:	98.0 percent of the target density.

Density measurements performed on Louisa bypass indicate that the compaction was consistently within the required range. Average void content of the in-place pavement was slightly less than 6 percent (7). Limited crushing of coarse surface particles occurred. Because of the coarse surface texture,

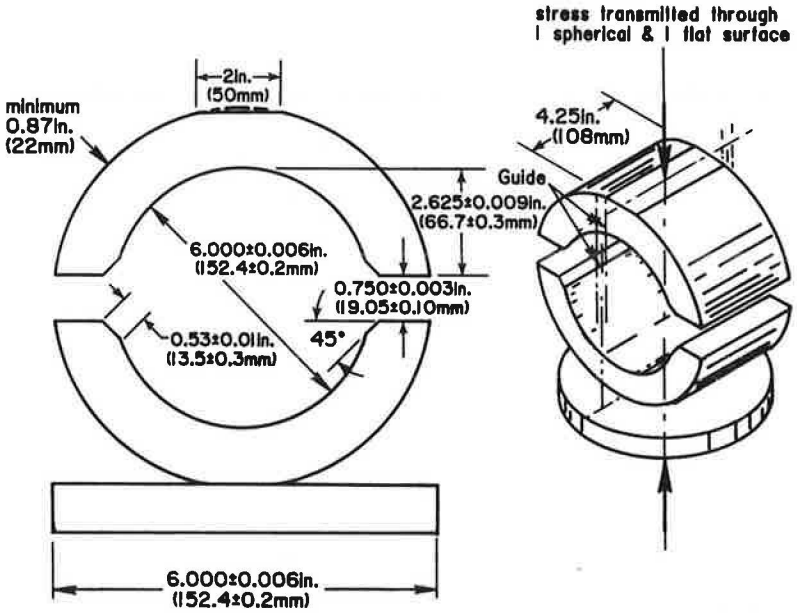

FIGURE 6 Breaking head.

TABLE 7 SUMMARY OF STABILITY AND FLOW RATIOS FOR LARGE-STONE MIXES

Agency (Year data obtained)	No. of Blows		Ratio	
	4"	6"	Stability	Flow
Penn. DOT (1969)	50	75	2.12	1.62
Penn. DOT (1970)	50	75	2.81	1.15
Penn. DOT (1988)	50	75	1.95	1.39
Penn. DOT (1988)	50	75	2.17	1.58
Penn. DOT (1989)	50	75	1.68	1.40
Jamestown Macadam (1989)	50	75	1.89	1.24
Kentucky DOH (1988) *	75	112	2.08	1.34
American Asphalt Paving (1989) *	75	112	2.37	1.63
American Asphalt Paving (1989) *	75	112	2.58	1.52
American Asphalt Paving (1989) *	75	112	1.98	1.68
American Asphalt Paving (1989) *	75	112	2.40	1.27
		No. of Mixes (N)	11	11
		Mean	2.18	1.44
		Std. Dev.	0.33	0.18

* Note : The average stability and flow ratio for these five mixes compacted with 75/112 blows are 2.28 and 1.49, respectively.

TABLE 8 TYPICAL MARSHALL MIX DESIGN DATA FOR 6-in.-DIAMETER SPECIMENS

Source : Kentucky Dept. of Highways. (Lawrence Co. - Louisa Bypass) Mix Type : Class K Base
Aggregates : Limestone #467 (55%), limestone #8 (20%), limestone sand (25%).
No. of Blows : 112 Asphalt : AC - 20
Design Gradation (% Passing) :

2"	1-1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
100	99	86	75	58	50	29	21	15	10	8	5	3.5

	% Asphalt Content				% Asphalt Content		
	3.2	3.6	4.0		3.2	3.6	4.0
Bulk Sp. Gr. (1)	2.424	2.410	2.440	Stability (1) (lbs)	5037	4980	4915
(2)	2.428	2.430	2.440	(2)	5683	5326	4627
(3)	2.419	2.434	2.437	(3)	5625	5236	5376
Mean	2.424	2.425	2.439	Mean	5448	5181	4973
Max. Sp. Gr.	2.546	2.530	2.515	Flow (1) (units)	17.5	14.5	14.0
% Air Voids	4.8	4.2	3.0	(2)	19.0	19.5	17.0
% VMA	11.4	11.7	11.6	(3)	17.0	14.5	15.0
% VFA	57.8	64.5	73.8	Mean	17.8	16.2	15.3

Remarks : AASHTO Gradations #467 (1-1/2" to #4) and #8 (3/8" to #8) were used. Stability values adjusted for specimen thickness.

nuclear densities were consistently lower than core densities taken at the same spot. The average nuclear density was about 1 lb/ft^3 less than core density, indicating that calibration is necessary for determination of actual values. A double-drum vibratory roller and a 25-ton pneumatic-tired roller (tire pressure up to 125 psi) were used for principal compaction.

The traffic is expected to densify the pavement to reduce air void content from about 6 percent as constructed to the design air void content (4.5 ± 1.0 percent).

Kentucky DOH provides a thin (1 in.) AC surfacing over Class K base to obtain a smooth and impermeable surface. Some technologists believe that ½-in.-thick hot sand and asphalt mix can also suffice. In any case, thicker surfacings should be avoided.

Field compaction data from projects in Kentucky and projects in Pennsylvania where large-stone mixes were used were provided by Kendhal (*5*). The test data indicate no significant problem in achieving compaction levels of >92 percent of the maximum mix specific gravity. Maximum aggregate size and lift thickness were 2 and 4 in., respectively, on Kentucky projects. Pennsylvania used 1½-in. maximum aggregate size and 2-in. lift thickness for the large-stone binder course mixes (*5*). All projects are reportedly performing satisfactorily, having been in service up to 2 years.

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

1. Premature rutting of low-volume roads used for hauling coal and logs is common. Use of large-stone asphalt mixes has been proposed to minimize the rutting potential of hot-mix asphalt used on these roads, which are subjected to heavy and channelized traffic. For the purpose of this report, "large stone" is defined as an aggregate with a maximum size of more than 1 in., which cannot be used in preparing standard 4-in.-diameter Marshall specimens.

2. A modified Marshall method for testing 6-in.-diameter specimens to accommodate large stones has been developed. The testing equipment is available commercially from two suppliers.

3. Statistical analysis of stability, flow, and air voids data indicate better repeatability of 6-in. specimens compared to 4-inch specimens when testing a large stone mix. The coefficient of variation for stability and flow values was reduced by at least 50 percent when the specimen size was increased.

4. The proposed method has the following significant differences from ASTM D1559–82 intended for testing 4-in. specimens:

   (a) Hammer weighs 22.5 lb. Only a mechanically operated hammer is specified.

   (b) Specimen size is of 6-in. diameter and 3¾-in. height.

   (c) Specimen usually weighs about 4050 g.

   (d) The mix is placed in the mold in two approximately equal increments, spading is specified after each increment.

   (e) The number of blows needed for 6-in.-diameter and 3¾-in.-high specimens is 1½ times the number of blows needed for 4-in.-diameter and 2½-in.-high specimen to obtain equivalent compaction levels.

   (f) A new table for stability correlations ratio needs to be used.

5. Comparative test data (4- versus 6-in.-diameter specimens) obtained from various highway agencies and producers indicate that the compaction levels are reasonably close.

6. Data obtained on stability ratio (stability of 6-in. specimen/stability of 4-in. specimen) and flow ratio (flow of 6-in. specimen/flow of 4-in. specimen) by various agencies was obtained and analyzed. The average stability and flow ratios were determined to be close to the theoretically derived values of 2.25 and 1.50, respectively. Therefore, it has been recommended that the minimum stability requirement for 6-in.-diameter specimens should be 2.25 times the requirement for 4-in.-diameter specimens. Similarly, the range of flow values for 6-in. specimens should be adjusted to 1½ times the values required for 4-in. specimens.

7. A typical mix design using 6-in. specimens for a coal-haul road is given.

8. The use of large-stone mix on coal haul roads in Kentucky has been described with limited data. It has been recommended to use a thin hot-mix asphalt surfacing over the large-stone asphalt mix to provide a smooth and impermeable surface.

ACKNOWLEDGMENTS

Cooperation of the following persons in supplying the relevant data and information is gratefully acknowledged: Larry Epley, Kentucky Department of Highways; David Allen, Transportation Center, University of Kentucky; Dean Maurer, Pennsylvania Department of Transportation; Ellis G. Williams, Consulting Engineer; Thomas Kerestes, American Asphalt Paving Company; and Thomas Olson, Jamestown Macadam, Inc.

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Evaluation of Weak Aggregates for Surface Dressing on Low-Volume Roads

M. E. WOODBRIDGE, P. A. K. GREENING, AND D. NEWILL

New roads in the Kalahari Desert region of Botswana must make best use of local materials if they are to be built economically. In the absence of rock outcrops, the main sources of roadstone are the variable duricrust deposits, described as calcrete or silcrete, which form in association with landform features such as pans or old river valleys. A study of four duricrusts for use as aggregate for surface dressing in lightly trafficked roads is described. Investigations included the determination of chemical and mineralogical composition, mechanical properties, and an assessment of the duricrusts' performance in full-scale road trials. Although generally described as calcrete or silcrete, the materials commonly contain mixed, disparate calcareous and siliceous fractions. In such cases, tests on representative samples of the whole material do not readily identify the influence of the weaker calcareous fraction on engineering properties. One of the road trials was designed specifically to examine the effect of varied proportions of calcareous and siliceous material on behavior under traffic. The results of the mechanical tests showed that with the exception of the one silcrete sample the materials were not strong enough to meet the strict Botswana specifications for surfacing aggregates. However, from the performance of the road trials and from a number of other countries where specifications are related to different traffic levels, new interim specifications were proposed in agreement with the Botswana Roads Department. On the basis of the recommended lowest traffic category of less than 0.8 million equivalent standard axles and 10 percent fines aggregate crushing test values of 130 and 100 kN for dry and soaked samples, respectively, it was considered that all of the materials investigated would be suitable for use in surface dressings for roads in the Kalahari Desert region.

The Transport and Road Research Laboratory (TRRL), United Kingdom, and the Ministry of Works, Transport and Communications (MOWTC), Botswana, are undertaking a joint research program in Botswana aimed at making best use of locally available materials for road construction. The work began because major new road projects, totaling some 1200 km, were planed to cross large areas of the Kalahari Desert, which covers more than three-quarters of the country (see Figure 1).

In the region, conventional sources of roadbuilding material, such as exposed rock, are scarce. Research, therefore, has concentrated on the main alternative sources—granular or gravelly materials known as calcrete or silcrete. These indurated materials, or duricrusts, are typical of those found in arid climates; by their process of formation within the soil profile, they are subject to considerable physicochemical variability.

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Earlier work in the research program (*1*) concentrated on calcretes and described the relationship among different types of calcretes, their properties, and the landform features where they occur. This work provided a basis for their identification and distribution. In another part of the work (*2*), calcretes of different quality have been and are continuing to be assessed in full-scale road experiments as base materials for lightly-trafficked bituminous-surfaced roads (*2*).

A study of duricrust materials from four different sources for use as surface dressing aggregate is described. Although the materials are referred to as calcretes or silcretes, some were of mixed composition. Because it was not known how this composition would affect the results of conventional aggregate tests or the duricrusts' performance in practice, special attention was placed on the relationship among physical, chemical, and petrographic properties. For one of the materials, additional subsamples were prepared by hand-sorting dominantly calcrete-rich and silcrete-rich fractions for inclusion in the testing program.

In assessing the results of the investigation, a comparison was made between the existing Botswana specifications for the use of surfacing aggregates, which have been adopted from the more demanding specifications of South Africa, and those of other countries that take account of lightly trafficked roads. This comparison, together with an assessment of a series of road trials carried out in Botswana and the United Kingdom using the materials in surfacing dressings, enabled recommendations to be made for the use of the Botswana aggregates.

DEFINITION AND DESCRIPTION OF CALCRETE AND SILCRETE

The calcretes and silcretes of Botswana are part of a wider group of materials described as duricrusts, which by pedogenic processes form as an indurated layer in the soil profile. Calcretes are characterized by the presence of calcium carbonate, although magnesium carbonate may also be present. Silcrete contains an accumulation of silica. Another important member of the group is ferricrete, or laterite, identified by high concentrations of sesquioxides (the oxides of iron and aluminum). The deposition of these materials is controlled by the solubility of the chemicals concerned, the original parent rock from which they were leached, the climatic and topographic environment, and the water table conditions. Calcretes and silcretes are distributed widely throughout the Kalahari and, although ferricretes also occur, these are confined to the eastern fringes closer to rock sources. A fuller definition of duricrusts is given by Goudie (*3*) and of calcretes

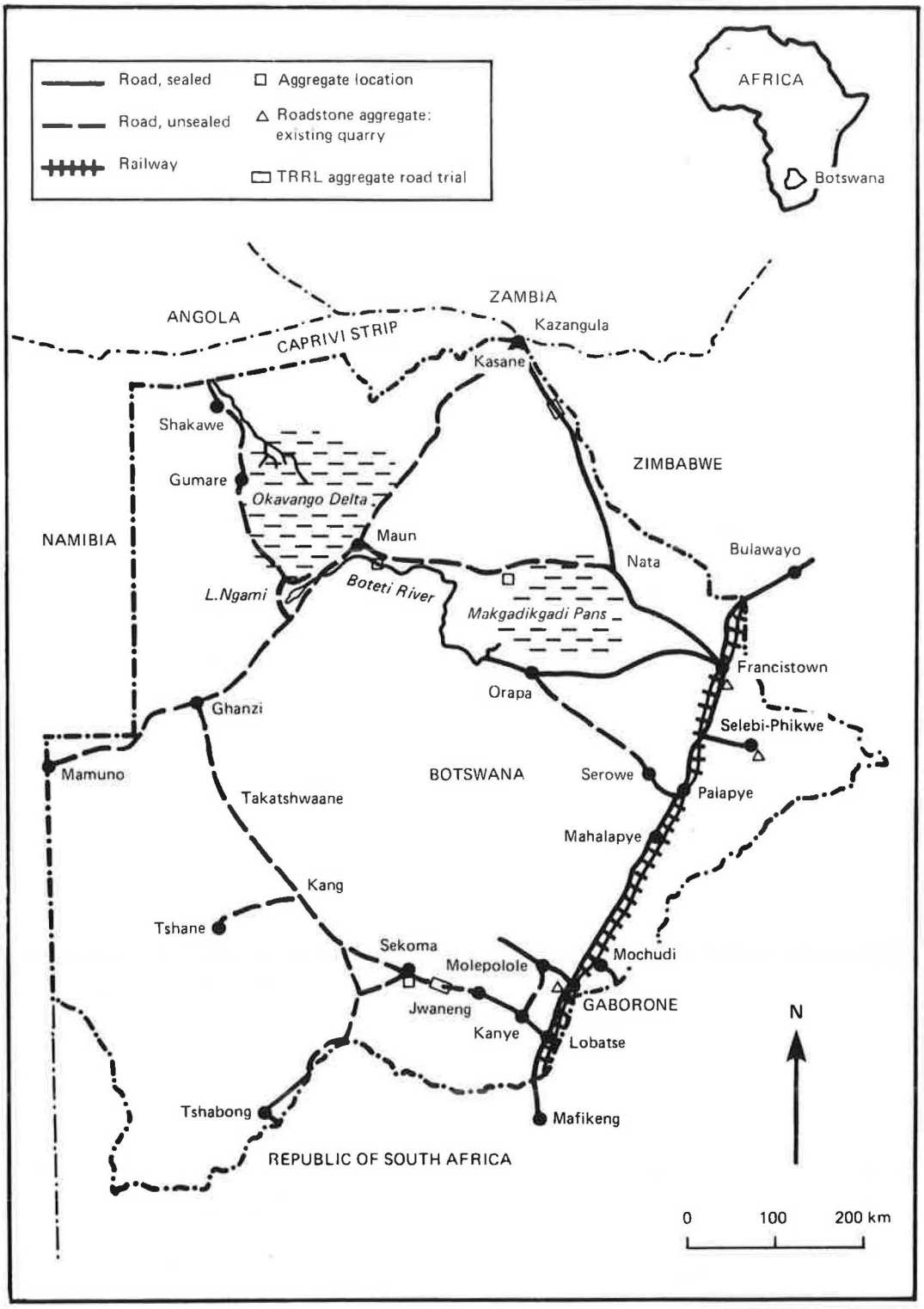


FIGURE 1 Republic of Botswana road network.

by Netterberg (*4*). From his work in southern Africa, Netterberg developed a geotechnical classification of calcrete related to an evolutionary sequence. Each type of calcrete can be recognized in the soil profile and represents a stage in the growth or weathering of a calcrete horizon with a significantly different range of geotechnical properties. The categories, in order of ascending evolutionary sequence, are as follows:

1. Calcified Soil. A soil horizon cemented by calcium carbonate to give a stiff consistency.
2. Powder Calcrete. Typically composed of loose silt or sand-sized carbonate particles, which when cemented together readily break down on working.
3. Nodular, Honeycomb, and Hardpan Calcretes. Mixtures of silt to gravel-sized, carbonate-cemented, host soil particles in a matrix of calcareous soil. The soil matrix is almost absent in the case of honeycomb calcrete and is completely replaced by hard carbonate cement in the case of hardpan calcrete.

Only the calcretes in Category 3 are hard and strong enough to be potentially suitable for surface dressing aggregate. The other categories are too soft or contain too much plastic clay-sized material.

A definition of silcrete provided by Grant and Aitchinson (*5*) states that, "silcrete is a siliceous material formed in a zone of silica accumulation in the earth's crust. The silica accumulation is produced by superficial physico-chemical processes and not by normal sedimentary, metamorphic, volcanic or plutonic processes." The main advantage of silcrete over calcrete is its potential for being harder and more resistant to abrasion. However, silcretes, like calcretes, are variable in quality depending on the degree of silicification and the nature of the material that is being replaced by silica. The silica is frequently chalcedony.

In Botswana, calcrete and silcrete are associated with a number of distinct landform features, such as interdune hollows, old river courses, topographic depressions, and the distinctive pans (enclosed depressions) of roughly circular shape. A loose relationship exists between landform and calcrete type, with the larger pans generally containing the higher quality nodular, hardpan, or honeycomb type of calcrete. Extensive work by TRRL (*1*) has led to a recommendation for the use of remote-sensing techniques, such as aerial photographs and Landsat imagery, to identify the geomorphological and landform features associated with calcretes. These techniques could be used at an early stage of road planning to help with materials surveys.

LOCATION AND DESCRIPTION OF SAMPLES

The two roads to be built to bitumen-surfaced standard associated with this aggregate study are the 300-km road from Maun to Nata in northern Botswana and the 650-km road from Jwaneng to Mamuno in the south. Both routes and the locations of the samples are shown in Figure 1.

For the Maun to Nata route, a reconnaissance survey that included borehole drilling of the best prospects was carried out along the proposed alignment. Those materials selected for the testing program were from km 175 from Maun (identified as the MN2 sample) and from the bed of the Boteti River at Samedupe, 13 km from Maun (identified as the Samedupe sample). The Nata sample came from a roadside quarry 9 km north of Nata. Numerous large pans likely to contain good quality calcrete or silcrete were located during the materials survey carried out between Jwaneng and Takatshwaane, which covers part of the area of the Jwaneng to Mamuno road project. The sample included in this study was from Sekoma pan (identified as the Sekoma sample). More detailed descriptions of the samples and the soil profiles are provided in the following paragraphs.

Maun-Nata (MN2) Sample

Figure 2 shows the location and borehole program undertaken to investigate the pan deposit, with typical borehole profiles. The material comprises thin, interbedded lenses of white calcrete and green/black silcrete in a fine-grained, sometimes soft sandy matrix. The upper 1 to 2 m is dense, whereas the lower layers become increasingly porous. Boreholes were drilled to a depth of 6 m on a 50-m grid and indicated an average thickness of 4-m stone of suitable quality. Potential gross reserves of the silcrete, assuming a density of 2.5 ton/m^3, were 0.2 million tons. The reserve of usable stone, which will depend on the proportion of acceptable quality for the surface dressing aggregate, is estimated to be 65 percent. Additional stone reserves may occur beneath or outside the drilled area. Because this was one of the materials containing a mixture of calcareous and siliceous fractions that could be distinguished by color, the samples for testing were hand-sorted into three parts. One was the run-of-quarry sample (comprising roughly equal proportions of calcrete and silcrete), and the other two were calcrete-enriched and silcrete-enriched fractions.

Samedupe Sample

The Samedupe material consists of abundant hard boulders of homogeneous quartzite occurring in and apparently limited to the bed of the Boteti River. Drilling failed to prove an extension of the quartzite beyond the river bank. The material is periodically extracted by local contractors for use as building stone and aggregate. Larger scale extraction may be inhibited by environmental considerations.

Sekoma Sample

The Sekoma material occurs in substantial quantities in the rim of a large pan. Following an exploratory borehole program, a pit 6 m deep was blasted near surface outcrops of hard calcrete. Disaggregated rubbly calcrete overlaid a hard calcrete breccia, comprising largely angular fragments of dark grey or reddish siliceous material in a light brown calcareous, sandy matrix. The breccia is the potential surfacing aggregate. In the base of the pit, unusable soft, calcareous, sandy material was found. Material from this source was used in a surface dressing trial constructed at Jwaneng in 1984.

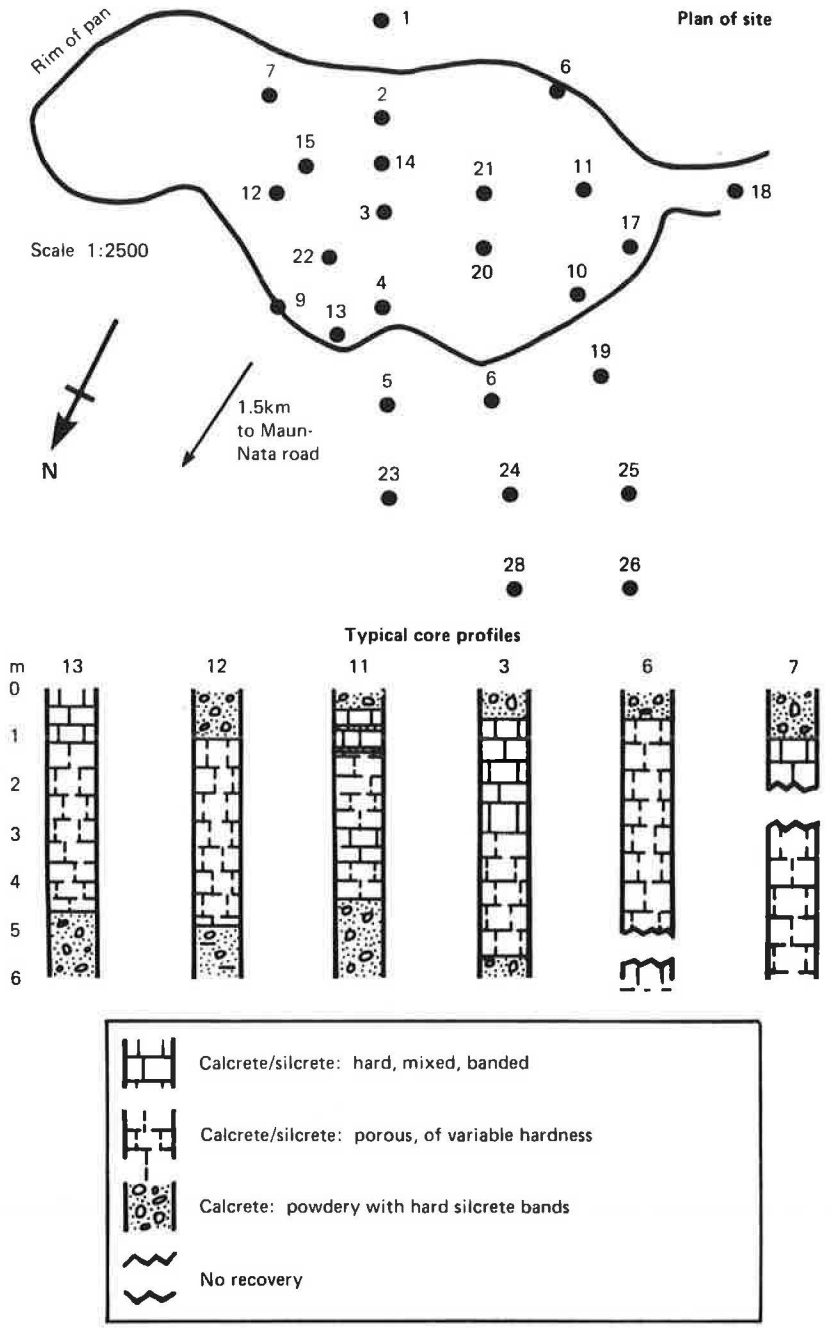


FIGURE 2 Borehole location and typical profiles at MN2 site on Maun-Nata Road.

Nata Sample

The Nata material is a fine-grained, hard, white-to-light-green sandy calcrete. It has already been used as roadbase for construction of the Nata-Kazangula Road, but the harder fraction was selected for a surface dressing trial at km 235 from Nata constructed in 1983. A 3-m face has been developed in the borrow pit, and the material has a rubbly appearance.

PETROLOGIC, CHEMICAL, AND MINERALOGICAL PROPERTIES

Visual examination indicated that, although the Samedupe and Nata materials were homogeneous, the MN2 and Sekoma materials were of varied composition. Therefore, the samples were examined more closely using a number of techniques. Thin sections were prepared and petrologically examined to

identify constituent mineralogy and fabric. Also, with the exception of the Nata sample, X-ray diffraction analysis, scanning electron microscopy, and microprobe techniques were used to examine certain zones of the samples and their microtexture before and after the accelerated abrasion and polishing tests. In addition, light emission spectrography and microprobe techniques were used for chemical analysis of these samples.

The results of the work are presented in Tables 1A, 1B, and 1C. The compositional differences among the samples are clearly indicated. The Samedupe sample was made up entirely of quartz grains welded together in a siliceous matrix, the Nata sample comprised quartz grains in a very-fine-grained carbonate matrix, and the MN2 and Sekoma samples included mixed but mainly quartz fragments in an essentially carbonate matrix. In the MN2 sample, individual fragments were strongly

TABLE 1A RESULTS OF PETROLOGIC TESTS—THIN-SECTION ANALYSIS (POINT COUNTING)

	MN2					
Component	Run-of-Quarry Stone (%)	Calcrete-Enriched Stone (%)	Silcrete-Enriched Stone (%)	Samedupe (%)	Sekoma (%)	Nata (%)
Carbonate	"	80	25	–	48	53
Quartz	"	12	63	99	35	42
Chalcedony	"	7	10	–	–	1
Olivine	"	–	–	–	10	–
Mica	"	–	–	–	5	–
Other	"	1	2	1	2	4

"Not determined.

TABLE 1B RESULTS OF PETROLOGIC TESTS—CHEMICAL COMPOSITION (EMISSION SPECTROGRAPHY AND MICROPROBE)

	MN2					
Component	Run-of-Quarry Stone (%)	Calcrete-Enriched Stone (%)	Silcrete-Enriched Stone (%)	Samedupe (%)	Sekoma (%)	Nata (%)
SiO_2 (median)	50.9	12.4	66.9	97.0	31.1	"
SiO_2 (range)	38–51	4–31	62–94	–	22–77	"
Al_2O_3 (median)	1.2	1.2	1.3	1.1	2.0	"
Al_2O_3 (range)	0.8–2.4	0–1.4	0.8–5.0	–	3–6	"
Fe_2O_3 (median)	0.5	0.5	0.6	0.5	1.3	"
Fe_2O_3 (range)	0.5–1.0	0.5–1.7	0.4–8.0	–	1–3	"
$CaCO_3$ (median)	39.6	80.1	22.7	0.3	57.6	"
$CaCO_3$ (range)	40–87	50–81	1–25	–	5–95	"
$MgCO_3$ (median)	10.2	5.6	12.3	0.2	6.0	"
$MgCO_3$ (range)	10–50	6–43	3–56	–	1–8	"
Na_2O_3 (median)	0.1	0.1	0.6	0.1	0.1	"
Na_2O_3 (range)	0–0.1	0.1–0.2	0–1.0	–	–	"
K_2O (median)	0.8	0.8	0.6	0.1	0.3	"
K_2O (range)	0–0.8	0.2–1.0	0–4.5	–	0.1–0.7	"

"Not determined.

TABLE 1C RESULTS OF PETROLOGIC TESTS—X-RAY DIFFRACTION AND MICROTEXTURE

Test	MN2	Samedupe	Sekoma	Nata
X-ray diffraction (CuK and radiation on nonoriented samples)	Alpha quartz, calcite, microcline felspar and dolomite	Alpha quartz	Quartz, calcite, felspar	Not determined
Microtexture (scanning electron microscope, after abrasion and polishing tests)	An overall decrease in roughness, with localized areas where the removal of large grains has increased roughness	An overall decrease of roughness caused by in-filling of the original pitted surface	An overall retention of roughness caused by removal of hard quartz grains	Not determined

differentiated into carbonate- or silica-rich constituents. Mineralogical and color differences coincided, the latter being used to sort the MN2 subsamples for the engineering tests, in which clear variations were established. The chemical analyses generally agreed with the composition determined by thin-section analysis.

In the examination of microtexture by the scanning electron microscope after the accelerated abrasion and polishing tests, the MN2 sample showed an overall decrease in roughness except in localized areas where quartz grains were plucked out to expose new surfaces. This plucking action appeared to be more dominant in the Sekoma sample. In the Samedupe aggregate, roughness decreased because of wearing of the original pitted surface.

SPECIFICATIONS FOR SURFACE DRESSING AGGREGATES

Surface dressing treatment is widely used in road maintenance to restore surface texture and resistance to polishing. It is also used to seal cracks in roads that are not structurally damaged. In many countries, double surface dressing treatments are used as the bituminous surface layer for lightly trafficked new roads. Assuming that good construction standards are achieved, surface treatments can be expected to last between 7 and 12 years, depending on the level of traffic and serviceability required.

Specifications for aggregates are based on a range of requirements relating to strength (resistance to crushing under traffic loading), durability (resistance to abrasive wear), resistance to the polishing action of traffic, and good adhesion to bitumen. In the present study, these properties were determined and initially compared to the current Botswana specifications for surface dressing stone (*6*), which are based on those used in South Africa (*7*). However, Botswana does not take into account different traffic levels, whereas a number of other countries do. A comparison was made, therefore, with neighboring countries in Africa and with Australia, the United Kingdom, and the United States, all of which permit a relaxation of specifications for low traffic volumes (see Table 2). Because the lower traffic volumes are more relevant to the design traffic levels for roads in Botswana, there clearly appears to be a need for more appropriate specifications that would allow a wider range of materials to be used. It was partly on this expectation that the present study was carried out.

Table 2 also shows that countries use different tests to specify aggregate requirements. In order to make comparisons among countries' specifications, use has been made of correlations that have been established between tests. For example, work by Shergold (*8*) and Minty et al. (*9*) showed good correlations between the aggregate crushing values, the 10 percent fines aggregate crushing values (10 percent FACT), and Los Angeles abrasion values, whereas Tubey and Beaven (*10*) and Hawkes and Hosking (*11*) demonstrated the correlation between the 10 percent FACT and the standard and modified aggregate impact values. The numbers in parentheses in Table 2 indicate where correlations have been made. To a certain extent, the degree of correlation is dependent on the rock properties being tested, although factors such as particle size and shape can also influence the results.

TABLE 2 COMPARISON OF SPECIFICATIONS FOR SURFACING AGGREGATES (FOR LIGHTLY TRAFFICKED ROADS)

Test	Botswana[a]	South Africa[b]	Kenya[c]	Zimbabwe[d]	Australia[e]	United States[f]	United Kingdom[g]
Aggregate crushing value (maximum, %)	(21)	21	26	30	(23)	(30)	–
10% fines aggregate crushing value (minimum, kN)							
Dry	210	210	(140)	120	(150)	(120)	–
Wet	160	160	(105)	90	(110)	(90)	–
Aggregate impact value (maximum, %)	(17)	(17)	(22)	(25)	(20)	(25)	–
Los Angeles abrasion value (maximum, %)	(21)	(21)	35	(40)	30–35	40–50	–
Aggregate abrasion value (maximum, %)	–	–	–	–	–	–	14
Polished stone value (minimum)	–	–	–	–	–	–	45
Flakiness index (maximum, %)	30	25–30	25	30	35	–	35
Sodium/magnesium sulphate soundness value (maximum, %, 5 cycles)	–	15	–	20	12	–	12
Adhesion to bitumen	>1[h]	>1[h]	–	–	–	–	<20[i]

[a]*Road Design Manual*, Ministry of Works, Transport and Communication, Botswana, 1982.
[b]*Technical Recommendations for Highways*, TRH 14, 1985.
[c]*Road Design Manual*, Part III, Ministry of Works, Transport and Communication, Botswana, May 1981 (less than 500 vehicles per day).
[d]*Zimbabwe: Part P*, Ministry of Roads and Road Traffic, Zimbabwe, Aug. 1973.
[e]*Principles and Practice of Bituminous Surfacing*, Vol. 1, National Association of Australian Road Authorities, 1984 (less than 300 vehicles per day).
[f]AASHTO 1986, M283-83.
[g]Department of Transport, Memo H176/76.
[h]Riedel and Weber Test (National Institute for Transport and Road Research, South Africa, 1979).
[i]Immersion Tray Test (Transport and Road Research Laboratory, United Kingdom, 1972).

PHYSICAL AND MECHANICAL PROPERTIES—TEST METHODS AND RESULTS

The engineering tests carried out, the methods used, and the results are presented in Table 3. The size of aggregate particles used in the tests was between 10 and 14 mm.

For weaker aggregates, the modified versions of the standard engineering tests are considered to be more appropriate because they accommodate the potential cushioning effect of the comparatively large amount of fine material produced during the tests. These tests, the modified aggregate impact test and the 10 percent FACT, can also be carried out on water-saturated, surface-dried samples to indicate their soundness. Unsoundness in aggregates is generally caused by weathering of primary rock minerals and is thus more applicable to igneous rocks. In sedimentary rocks, however, the presence of microcracks and pores may also be an indication of unsoundness. Thus, soundness tests were used to evaluate the inherent weakness of the material rather than its weathering potential. Normally, a petrologic test indicates soundness, but a chemical test using a saturated solution of sodium or magnesium sulphate is widely employed. This test, however, has poor reproducibility, especially with the sodium sulphate salt, which has several states of crystallization. It is also reported (*12*) that carbonate rocks are susceptible to chemical attack by the salt solution.

TABLE 3 RESULTS OF PHYSICAL AND MECHANICAL TESTS

	Maun-Nata (MN2)											
	Run-of-Quarry Stone		Calcrete-Enriched Stone		Silcrete-Enriched Stone		Samedupe		Sekoma		Nata	
Test and Method	Dry	Soaked	Dry	Soaked	Dry	Soaked	Dry	Soaked	Dry	Soaked	Dry	Soaked
Aggregate crushing value (%) (BS812:1975, Part 3)	23	27	24	31	23	24	19	17	18	22	20	22
10 percent fines aggregate crushing value (kN) (BS812:1975, Part 3)	180	120	160	80	200	170	240	250	220	140	200	170
Aggregate impact value (%) (BS812:1975, Part 3)	25	25	25	29	20	24	20	23	20	22	21	21
Modified aggregate impact value (%) (Hosking and Tubey, 1969)	30	33	33	36	27	24	37	30	–	–	22	24
Aggregate abrasion value (%) (BS812:1975, Part 3)	2.0	–	6.3	–	1.4	–	1.7	–	6.4	–	6.0	–
Polished stone value (BS812:1975, Part 3)	46	–	–	–	–	–	48	–	57	–	Not tested	
$MgSO_4$ soundness test (%) (ASTM C88)	0.2	–	0.8	–	0.4	–	0	–	6.5	–	Not tested	
Water absorption test (%) (BS812:1975, Part 3)	1.1	–	2.3	–	1.0	–	0.6	–	2.4–3.5	–	1.4	–
Flakiness index (%) (BS812:1990, Section 105.1)	27	–	25	–	30	–	50	–	25	–	15	–
Static immersion adhesion test at 40°C using MC30 (*Bituminous Materials in Road Construction*), TRRL, 1972)	No reaction	–	–	–	–	–	5%	–	No reaction	–	Not tested	

In order to assess the adhesion of aggregate to bitumen, the Riedel and Weber test (*13*) was attempted. It was not possible to obtain consistent results even using aggregate known to have good adhesion properties. The immersion tray test (*14*) was therefore used in which a visual assessment was made of stone coated with different binders and immersed in water at 25°C and 40°C for periods from 1 hr to 8 days. The results presented are those with MC 3000, a cutback binder commonly used in Botswana. The samples were immersed for 24 hr at 40°C.

The results of the mechanical tests showed that, if the stricter specifications of the Botswana Road Design Manual are applied (on the basis of the 10 percent FACT test), then only the Samedupe material met the requirement for surface dressing aggregate. The Nata and Sekoma materials were marginal, and MN2 was below the requirement.

The only other country whose specifications for aggregate strength are based on the 10 percent FACT test is Zimbabwe (*15*), which sets minimum values of 120 and 90 kN, respectively, for tests on dry and soaked samples. All of the aggregates satisfied these criteria. It is interesting, however, to consider the MN2 and Sekoma samples, which as mixed aggregates had relatively high proportions of calcrete. The presence of this calcareous fraction resulted in a considerable reduction in strength when the samples were saturated in water, more than the allowable 25 percent difference between samples tested in the dry and soaked condition. The effect of the weaker calcrete fraction was clearly demonstrated in the tests on the separated fractions of MN2 aggregate. A 50 percent loss of strength (from 160 to 80 kN) was obtained for the calcrete-enriched stone compared with 15 percent for the silcrete-enriched stone. The loss of strength on soaking the mixed material was 33 percent (from 180 to 120 kN). These results indicated that, for variable samples, tests on notionally representative samples may not truly reflect performance because of the disparate nature of the constituents. The application of normal specifications for acceptance of the material is also made difficult. It was mainly for this reason that a separate laboratory investigation was undertaken in which a range of engineering tests was carried out on mixtures of rock comprising different proportions of relatively hard and soft U.K. aggregates (flint and limestone). This work is reported elsewhere (*16*) but, in general, it was shown that there was a linear relationship between the aggregate strength and the ratio of hard to soft particles.

Other tests on the MN2 samples, such as aggregate abrasion, magnesium sulphate soundness, and water absorption, also reflected the difference between the calcrete and silcrete fractions, although the abrasion values were well within the U.K. Department of Transport specifications for lightly trafficked roads. The other samples also met this requirement. Further tests on the samples, not including the Nata aggregate, showed that they satisfied the minimum value of 45 in the Department of Transport's specification for polished stone value. They also showed satisfactory adhesion to bitumen. The Samedupe sample, although being a strong and sound material, failed the flakiness specification. It may be possible, however, to modify the crushing technique to increase the proportion of nonflaky stone.

The engineering tests showed that care has to be taken when applying test results of aggregates of mixed composition. To deal with these problems, it was clear that further assessments based on performance in practice were important. Thus, a series of road trials was undertaken as part of the study; two of these were in Botswana and one was in the United Kingdom.

ROAD TRIALS

The first Botswana trial, constructed in June 1983 on the Nata-Kazangula Road, used the Nata material; the second, constructed in June 1984 close to Jwaneng, used the Sekoma material (see Figure 1). The materials were laid in various sections as single and double seals. In the Nata-Kazangula trials, the underlying roadbase material was a basaltic gravel. In the Jwaneng trial, two different roadbase materials were used: one was a plastic calcified sand (Kalahari sand loosely consolidated in a carbonate matrix) and the other was a nodular calcrete gravel. These are materials likely to be used in the Kalahari region. In this trial, graded aggregates as well as single-sized aggregates were used as the surfacing material.

The performance of the surface dressing trials was subjectively assessed and is summarized in Table 4. The Nata material has performed particularly well, especially the double surface dressings, which have not received any maintenance to date. The Jwaneng trial with the Sekoma material has not performed as well; generally, there was an underapplication of a low viscosity binder due to the absorptive calcrete roadbases. The Sekoma aggregate, which had water absorption values between 2.4 and 3.5, also probably absorbed binder. As a result, most of the single seals and the top layer of the double surface dressings were lost in this trial. Inspection of the longer term performance of the aggregate in the double seals indicated that some of the softer calcrete particles were cracked and abraded, although generally the surface dressing matrix remained in place. The performance of the graded seals compared with the single seals plus crusher fines was obscured by the problem with binder application. Since this road trial, however, the Botswana Ministry of Works has been carrying out further work using graded seals. The traffic levels in the Botswana trials, between 100 and 150 vehicles per day, were what might be expected on trans-Kalahari routes.

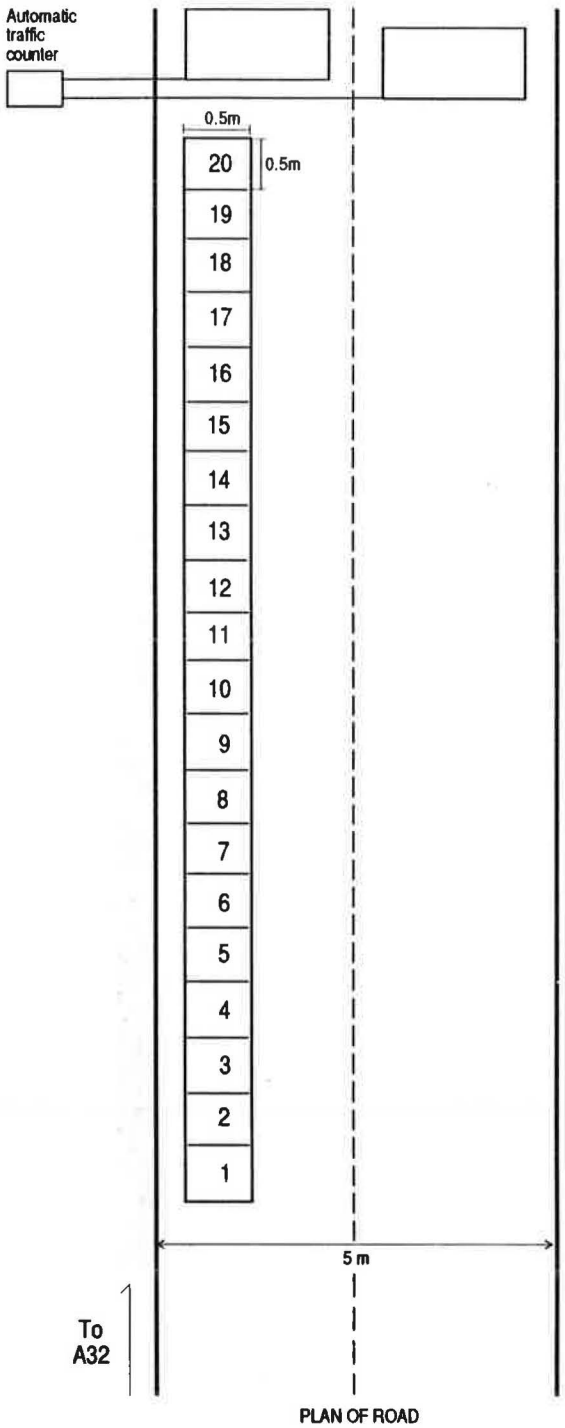
In order to assess the other aggregates included in the study—the MN2 and Samedupe materials—a pilot scale trial was constructed in August 1989 near Winchester in the United Kingdom on a minor road whose premix macadam wearing course was being surface dressed with a single seal. This action was part of a normal periodic maintenance operation. Twenty small sections, each 0.25 m^2, were laid in the verge side wheeltrack. Sekoma aggregate was included, as well as the Samedupe and MN2 aggregates. For the MN2 materials, some sections comprised different proportions of calcrete- and silcrete-rich material. Several mixtures of the U.K.-derived flint and soft limestone were also included. The layout of the trial sections is shown in Figure 3. A cutback bitumen binder was applied at a rate of 1.1 L/m^2 for all sections. Although this rate was the design rate for the control stone, which was of 10-mm nominal size, several of the experimental sections were of 14-mm size. Stone application rate was 10 kg/m^2.

Traffic on the road is 700 vehicles per lane per day, and measurements to date have recorded 170,000 vehicle passes over the sections in an 8-month period. This traffic level is

TABLE 4 SURFACE DRESSING ROAD TRIALS IN BOTSWANA

Section	Road base	Surface Aggregate	Surface Treatment (Single or Double Seal, Stone Size)	Binder Application Rate (L/m^2)		Stone Application Rate (kg/m^2)		Traffic (1987 veh/day)	Maintenance Treatment	Performance Rating Scale After 5 Years[a]
				1st Seal	2nd Seal	1st Seal	2nd Seal			
Nata-Kazangula Road Trials (Constructed June 1983)										
1	Basalt gravel	Nata calcrete	13 mm	1.6	–	12	–	140 (52% > 5 t.)	Slurry seal (Nov. 1986)	4 to 1
2			13 mm, with fog spray	1.6	–	12	–			3 to 1
3			13 mm + crusher waste	1.6	1.2	12	12			2
4			13 mm + 6.7 mm double	1.6	1.1	12	5		No treatment	2
Jwaneng-Sekoma Road Trial (Constructed June 1984)										
1[b]	Calcified sand	Dolerite[c]	19 mm + 6.5 mm	1.5	1.5	19	11	112 (34% > 5 t.)	Fog spray (July 1985)	3
2			19 mm + 6.5 mm	2.1	1.6	19	11			3
3			19 mm + fines	2.7	0.9	19	12			3–4
7		Sekoma calcrete	Double graded seal[d]	1.4	1.5	30	24			3
15	Nodular calcrete		13 mm + fines	1.6	1.2	12	12			3
17			19 mm + 9.5 mm	1.8	1.4	19	11			4
18		Dolerite[c]	19 mm + 9.5 mm	1.6	2.0	19	11			3

[a]Performance rating based on TRH 6, South Africa, 1985, Table V, p. 29. 1 = no discernible loss of stone; 2 = same as 1 with assessed hardening of binder; 3 and 4 = progressive loss of stone from top layer of multiple seals; 5 = loss of stone from all layers of multiple seals. Cutback bitumen binder used in both trials.
[b]Sections 4, 5, 6, 8–14, and 16 of the Jwaneng-Sekoma road trial were all single seals and were lost within 5 years.
[c]The dolerite aggregate used was a control, with 10 percent FACT values of 240 to 270 kN.
[d]Double-graded seal of Section 7 comprised 19- to 3-mm and 14- to 3-mm aggregate.



Date of Inception: 9 August 1989
Location: O.S.Ref.632238
Traffic: ca.700vpd per lane

EXPLANATION OF TRIAL SECTIONS

Section no.	Material	Description	Stone size (mm)
20	Control (UK)	Basalt	-10 + 6.7
19	Samedupe (Botswana	Silcrete	-14 +10
18	Sekoma (Botswana)	Calcrete breccia	-14 +10
17	MN2; Silcrete-calcrete (Botswana)	100% Calcrete; 0% silcrete	-14 +10
16		50% Calcrete; 50% silcrete	-14 +10
15		0% Calcrete; 100% silcrete	-14 +10
14	UK aggregates	100% Limestone; 0% flint	-14 +10
13		75% Limestone; 25% flint	-14 +10
12		50% Limestone; 50% flint	-14 +10
11		25% Limestone; 75% flint	-14 +10
10		0% Limestone; 100% flint	-14 +10
9	MN2 (Botswana)	100% Calcrete; 0% silcrete	-10 + 6.7
8		0% Calcrete; 100% silcrete	-10 + 6.7
7	Sekoma (Botswana)	Calcrete breccia	-10 + 6.7
6			-10 + 6.7
5	MN2 (Botswana)	100% Calcrete; 0% silcrete	-10 + 6.7
4		75% Calcrete; 25% silcrete	-10 + 6.7
3		50% Calcrete; 50% silcrete	-10 + 6.7
2		25% Calcrete; 75% silcrete	-10 + 6.7
1		0% Calcrete; 100% silcrete	-10 + 6.7

Notes:
Binder type: 'cutback' bitumen
Quantity: 1.1 litres /m²
Stone application rate: 10kg/m²
Section 20 is the same material as applied to the rest of the road surface on 9 August

North (approx)

FIGURE 3 Surface dressing trial layout at Ropley, near Winchester, United Kingdom.

about 10 times more than would be expected on trans-Kalahari roads and is therefore approaching the total traffic expected during a 7- to 10-year design life of a surface dressing. So far the sections have performed well with the exception of the two Sekoma sections, where approximately 20 percent of stone has been lost. Even the calcrete-enriched fractions of MN2 laid in Sections 5, 9, and 17, which have lower aggregate strengths than the Sekoma material, have performed better. So has the U.K. soft limestone in Section 14. Satisfactory performance has been obtained from the larger sizes laid in Sections 10 to 19 inclusive (with the exception of the Sekoma stone), even though the bitumen application rate was designed for the 10-mm size.

At this stage, it is probably premature to rate the Sekoma aggregate as unsatisfactory because in practice it would be laid in new construction as a double surface dressing and not as a single seal. Also, in the wintry conditions of the U.K. trials, the wetness and frosts are more severe than would be experienced in Botswana. However, further evidence of the weakness of the Sekoma aggregate was shown by the results of the mechanical tests, in which the water absorption, abrasion loss, and magnesium sulphate soundness values were higher than the other aggregates examined.

CONCLUSIONS AND RECOMMENDATIONS

The wide variation in the types of duricrust occurring in the Kalahari region of Botswana means that careful attention must be paid to methods of identification to recognize deposits likely to be suitable as aggregate for use in road construction. Even among the harder materials, the mode of deposition requires that cores from drilling have to be examined to estimate the quantities or reserves of material available. Four different duricrusts identified as potential surface dressing aggregates for major new road projects were investigated in this study by examining their composition, mechanical and engineering properties, and performance in road trials.

Although the materials were initially classified as calcretes or silcretes, they were more typically of mixed composition. The calcareous and siliceous fractions were variable and could strongly influence properties and behavior. Examination of the separated fractions of one of the aggregates (MN2 sample) showed that the calcareous portion was weaker and susceptible to further softening when saturated with water. The disparate nature of individual fractions makes it more difficult to assess test results of representative samples of the whole material.

In order to determine the mechanical strength of weaker or marginal quality aggregates, it is important to use the now generally recognized modified forms of standard tests such as the 10 percent FACT or modified aggregate impact test. In this study, the 10 percent FACT test was more discriminating, although the apparatus for the aggregate impact test is simpler, cheaper, and portable. Other tests that can be used to differentiate weaker aggregates are the aggregate abrasion test and water absorption.

All of the aggregates included in the study were incorporated into road trials. The Sekoma aggregate performed less well, but there is sufficient evidence to indicate that they would all be satisfactory as surfacing aggregates for lightly trafficked roads. This is especially the case for new roads when double surface treatments are used. The use of graded aggregate seals as opposed to using single-sized chippings and the use of a slurry seal in the second application may also provide better protection of exposed aggregate particles. These construction practices are currently being examined by the Roads Department in Botswana together with a wider range of test methods on other marginal materials.

The results of the study and the comparison made of different specifications used by other countries for surfacing aggregates have shown clearly that the existing Botswana specifications were too stringent for lightly trafficked roads. Following discussions with the Botswana Roads Department on the results of the work and evidence from other work in Botswana, new interim specifications have now been proposed, which are set out below. (Generally, values of tests on soaked samples should be 75 percent of those on dry samples. However, this requirement may be relaxed for roads constructed in the drier regions of the Kalahari provided that the minimum soaked test value is satisfied.

Minimum 10% FACT Values (kN)		*Pavement Design Category [equivalent standard axles (esa)]*
Dry Test	*Soaked Test*	
180	135	>3 million
150	115	0.8–3 million
130	100	<0.8 million

ACKNOWLEDGMENTS

This paper is published with permission of the Director of the Transport and Road Research Laboratory and Her Britannic Majesty's Stationery Office, United Kingdom, and the Roads Department of the Ministry of Works, Transport, and Communications, Botswana. Assistance in materials testing was provided by Queen Mary College, London, and in the U.K. road trial by Hampshire County Council. The work described here is part of the program carried out for the Overseas Development Administration.

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Views expressed herein are not necessarily those of the British government.

Stabilization of Alluvial Soils with Cement and Cement–Rice Husk Ash Blend for Low-Volume Road Construction in Bangladesh

A. S. M. MUSTAQUE HOSSAIN, ALAMGIR M. HOQUE, AND JOHN P. ZANIEWSKI

Rice husk, produced by milling rice, has been used to a large extent as boiler fuel in rice mills in Bangladesh. The rice husk ash (RHA) produced has created a disposal problem. A study was conducted to examine the characteristics of alluvial silty soils stabilized with ordinary portland cement and portland cement–RHA blend and to assess their applicability for low-volume road construction in Bangladesh. Stabilized samples were prepared at maximum dry density and optimum moisture content determined by AASHTO T99. The samples were cured and tested for durability, volume and moisture change characteristics, unconfined compressive strength, and plasticity. Cement-treated alluvial soils satisfy the durability criteria recommended by the Portland Cement Association (PCA) at about 9 percent cement content. At this cement content, however, they do not attain the specified minimum unconfined compressive strength. Silty soils stabilized with only 2 percent cement content show considerable gain in unconfined compressive strength over untreated soil. RHA can be blended with cement to stabilize silty soils. A partial replacement of cement by as much as 25 percent of ash by weight is possible without impairing durability or appreciably decreasing strength compared with samples containing cement only. RHA addition results in an increase in volume on wetting of the soil-cement-RHA mixture and decreases the maximum dry density of the soil. Higher ash content results in an increase in the plasticity of the cement–RHA-stabilized soil.

Bangladesh has a main highway network of 6,770 mi, including approximately 2,900 mi of unpaved roads. Because of low topography, roads are built on fill soils from nearby borrow pits. Most unpaved roads are minimally compacted earth roads built on fill soil that serve the rural communities and provide connections with the regional growth centers. Traffic volumes on these roads are very low, consisting mostly of animal-drawn wagons carrying farm produce and occasional motorized vehicles. Annual maintenance practices include dumping loose soil from nearby borrow pits over the road formation in winter and rendering nominal compaction, usually by manual methods. Due to limited resources, state-of-the-art compaction equipment and quality control methods are not applied for this type of secondary road construction. These roads subsequently are exposed to rain and monsoon floods. These environmental factors, together with inadequate compaction, seriously impair their durability. Most of the roads become impassable in the rainy season. For paved-road construction, greater pavement thickness is required because of the weak subgrade material. Cement stabilization of local soils has been suggested as an alternative way to improve the characteristics of borrow and in-place soils in earth and paved-road construction (*1*). The cost savings achieved by stabilized earth roads is well established (*2*).

In a developing country such as Bangladesh, it is extremely difficult to mobilize resources for road construction. Use of local materials is an important prerequisite for proper and economical road construction. Rice husk ash (RHA) is an agricultural waste produced by the rice boilers. In Bangladesh, annual rice production is around 13.5 million tons, resulting in a considerable amount of ash. RHA has potential as a construction material similar to pulverized fuel ash (PFA) produced in pulverized coal-fired electricity-generating plants.

Cement has long been used in soil stabilization. The use of RHA with lime as a stabilizer has been investigated and found successful (*3,4*). A limited study in Bangladesh (*5*) has demonstrated that a portland cement–RHA blend in a 1:1 ratio can be used in masonry work satisfying ASTM specification C91 for masonry cement. Ahmed (*6*) investigated the geotechnical properties of some Bangladeshi silty soils stabilized with cement and lime. However, the quality of lime in Bangladesh is poor and shows a high variability in properties. Also, no previous work has been done on cement-RHA stabilization of soils. An experimental analysis was required to establish the applicability of cement-RHA stabilization for alluvial silty soils in Bangladesh.

SOILS OF BANGLADESH

The surfacial geology of Bangladesh can be split into three formations: (a) tertiary and pleistocene hill formations, (b) uplifted alluvium terraces, and (c) recent floodplain and piedomont alluvium. Of these, recent floodplain and piedomont alluvium occupies nearly 70 percent of the land area. The floodplain deposits are of recent origin, and soils alternate in

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repeated layers of clays, silt, and sands. Rivers in this formation are flat in gradient and deposit fine soil particles, predominantly silt and clay, every year. Major portions of the deposits are inundated by seasonal flooding. As a result, the subsoil becomes soft and has low density and shear strength. The high groundwater table at other times of the year also contributes to the low density and bearing capacity of the subsoil. The strength of these soils must be improved for road construction.

MATERIAL CHARACTERISTICS

Silts are abundant in the floodplain alluvium. The soils in this formation do not vary widely in their index properties and textural compositions (*1*). Two representative silty soil samples from this formation were selected for this study, one with no plasticity and the other with little plasticity. Table 1 presents the properties of untreated soils, and Figure 1 shows the grain size distribution of these soil samples. Both soils are classified as AASHTO A4. Type I portland cement meeting ASTM standards was used as the primary stabilizing agent.

RHA was produced by burning rice husk under controlled temperatures of 500°C to 525°C in the laboratory. The burning was discontinued after visual identification of white ash, which signified low carbon content. The ash was ground in a ball mill and sieved to pass through a No. 200 sieve. The chemical composition of the RHA was then analyzed, and the results are presented in Table 2.

MECHANISM OF SOIL-CEMENT STABILIZATION

Addition of cement has a twofold effect on fine-grained silts and clay: (a) acceleration of flocculation and (b) promotion of chemical bonding. Because of flocculation, the clay particles are electrically attracted and aggregated with each other. This attraction results in an increase in the effective size of the clay aggregation, converting it into the mechanical equivalent of the silt (*7,8*). The chemical bond aggregates the particles in a cellular structure. Because flocculation binds the clay particles in a matrix form, the cement reduces plasticity and increases shear strength. The chemical surface effect of the cement reduces the water affinity of the clay and subsequently the water-retention capacity of the clay. This action results in the enclosure of the larger unstabilized grain aggregates, which, cannot expand and thus gain improved durability.

The cement-clay interaction is significantly affected by the interaction of lime, produced during hydration of cement and the clay minerals. The interaction can be classified into two groups: (a) rapid rate (ion exchange and flocculation) and (b) slow processes (carbonation, pozzolanic reaction, and the production of new substances). The products of rapid rate processes harden into high-strength additives, whereas the slow

TABLE 1 PROPERTIES OF UNTREATED SOIL

Soil Property	Soil - A	Soil - B
Textural Composition: (MIT Classification)		
Sand, % (2 mm - .06 mm)	14	6.5
Silt, % (.06mm - .002mm)	86	89.5
Clay, % (<.002mm)	0	4.0
Percent passing # 200 sieve	95	98.0
Materials smaller than 0.05 mm, (%)	81	84.0
Atterberg limits and indices:		
i) Liquid limit	-	33.0
ii) Plastic limit	-	27.5
iii) Plasticity Index	-	5.5
Natural moisture content	26	23.0
Specific gravity	2.63	2.68
Engineering Properties:		
Optimum moisture content, % (AASHTO T99)	15	18
Maximum dry density, pcf	98.5	104.6
Unconfined Compressive Strength, psi	9.67	11.23
Chemical Properties:		
pH	7.2	6.6
Organic matter content, %	0.71	0.62
Classification:		
AASHTO	A-4	A-4(0)
Unified/ASTM	ML	ML
General Rating as Subgrade:		
AASHTO	Fair to Poor	Fair to Poor
Unified/ASTM	Not Suitable	Not Suitable

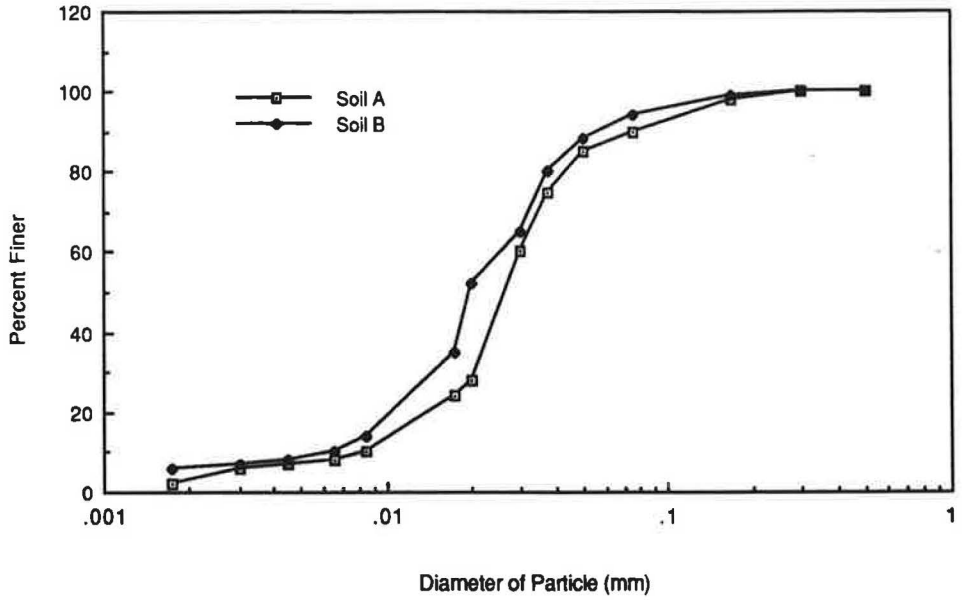

FIGURE 1 Grain size distribution curves of tested soils.

processes increase the strength and durability of the soil-cement mix by producing an additional cementing substance to further enhance the bond strength between the particles (*2*).

ROLE OF RHA IN STABILIZATION

Chemical analysis of RHA indicates the presence of silica as a primary constituent. The silica may be present in the ash in amorphous form. The combination of amorphous silica in RHA with calcium hydroxide, either as slaked lime or as a byproduct of hydrating cement, results in a cementing agent such as pozzolanic cement (*9*). The fine ash also acts as a source of reinforcement in the set cement (*10*).

TEST PROGRAM

Soil samples were selected for research on the basis of index properties, pH, and organic matter content recommendations of the Indian Road Congress (*11*) for soil-cement stabilization. Samples were compacted to find optimum moisture content and maximum dry density by AASHTO T99. The samples were tested for unconfined compresssion at the maximum dry density. Next, the soils were stabilized with cement using cement contents of 2, 4, 6, 8, and 10 percent by weight of air-dried soil and subjected to a wetting and drying test. Of the cement contents used, the minimum cement content required to satisfy the soil-cement loss criteria specified by the Portland Cement Association (PCA) (*12*) was 10 percent for both soils. The stabilized samples with cement contents of 2, 8, and 10 percent were tested for unconfined compressive strength. The 2 percent cement content was selected to assess the strength characteristics of cement-modified alluvial silty soils, whereas the other two cement contents were selected on the basis of recommendations by Catton (*13*) for AASHTO A4 soils.

The soils were later stabilized with admixtures of portland cement and RHA blend at optimum moisture content, keeping the total amount of admixture equal to 10 percent. Cement was replaced in the total admixture by RHA in proportions, cement to RHA, of 3:1, 2:1, and 1:1 by weight. These cement–RHA-stabilized samples were tested to evaluate plasticity characteristics, durability, and unconfined compressive strength by the Atterberg limits test, wetting and drying test, and test for unconfined compression, respectively. Details of the tests

TABLE 2 CHEMICAL COMPOSITION OF RHA

Constituent Present	Percent by Weight
SiO_2	91.08
Al_2O_3	0.56
Fe_2O_3	0.60
CaO	1.23
MgO	1.30
Loss on ignition	5.23

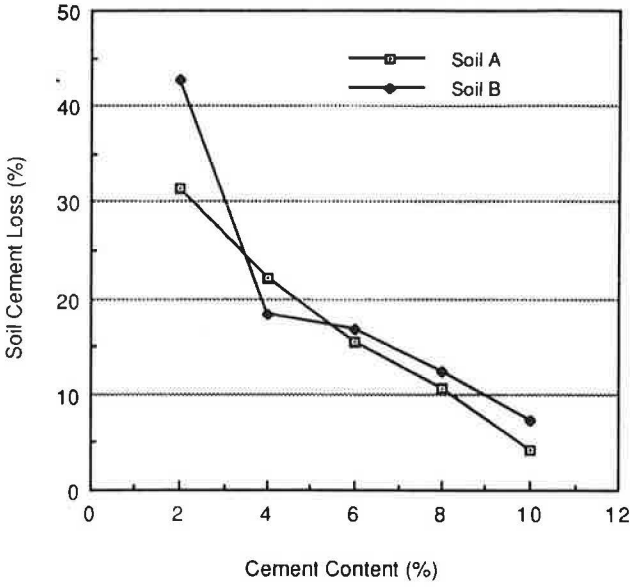

FIGURE 2 Effect of admixture content on soil-cement loss of soils in wet-dry test.

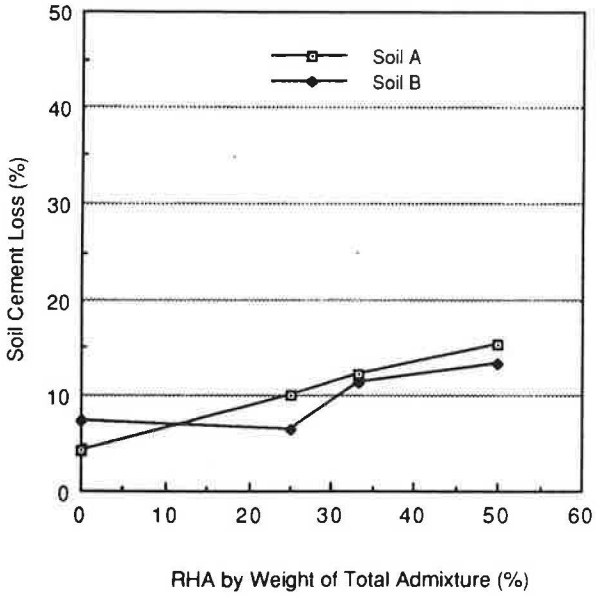

FIGURE 3 Effect of replacement of cement by RHA in the total admixture on soil-cement loss of soils.

along with the standard methods followed can be found elsewhere (*14*).

TEST RESULTS, ANALYSIS, AND DISCUSSIONS

Minimum Cement Content

The wet-dry tests were conducted on cement and cement–RHA-stabilized soils to assess the durability, moisture, and volume change of the stabilized soils. The samples were subjected to 12 cycles of wetting and drying. Tests were done on two sets of samples, one for finding volume and moisture change and one for determining soil-cement loss.

The test results were used to determine the minimum cement content required for soil-cement mixtures to satisfy the PCA soil-cement loss criteria. Figure 2 shows the relationships between soil-cement loss and cement content for Soils A and B. Higher cement contents reduce the soil-cement loss in the wet-dry test. Cement contents of 8.1 and 9.0 percent for Soils A and B, respectively, meet the PCA criteria of a maximum of 10 percent soil-cement loss for AASHTO A4 soil. These cement contents also satisfy the PCA recommendation of 7 to 12 percent cement for stabilization of AASHTO A4 soils.

Figure 3 shows the relationships between the soil-cement loss in the wet-dry test and the amount of RHA. As shown, the soil-cement loss increases with increasing ash replacement of cement. The amount of cement that can be replaced by ash is 25 percent for Soil A and 31 percent for Soil B to meet the PCA criteria for soil-cement loss in the wet-dry test. Soil B has a 4 percent clay content, which can be used to explain the higher ash replacement of cement for Soil B than Soil A corresponding to a 10 percent soil-cement loss.

Moisture Change

Maximum moisture content is the highest amount of water in the stabilized sample during wet cycles of the wet-dry test. Figure 4 shows the maximum moisture contents for Soils A and B versus cement contents. The maximum moisture content occurs at 10 percent cement content. Figure 5 shows the maximum moisture contents in Soils A and B when stabilized with RHA and cement. From this figure, it is evident that RHA addition has little effect on maximum moisture content.

Volume Change

The relationships between volume change and percent of cement content for Soils A and B are shown in Figure 6. Increasing cement content produces shrinkage in both soils due to shrinkage during the cement hydration. Addition of RHA decreases shrinkage, as shown in Figure 7.

Unconfined Compressive Strength

The relationship among unconfined compressive strength, cement content, and curing period are shown in Figures 8 and

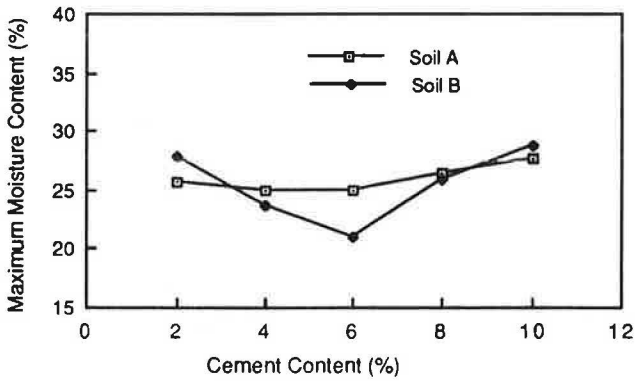

FIGURE 4 Maximum moisture content in cement-stabilized soils during wet cycles of wet-dry test.

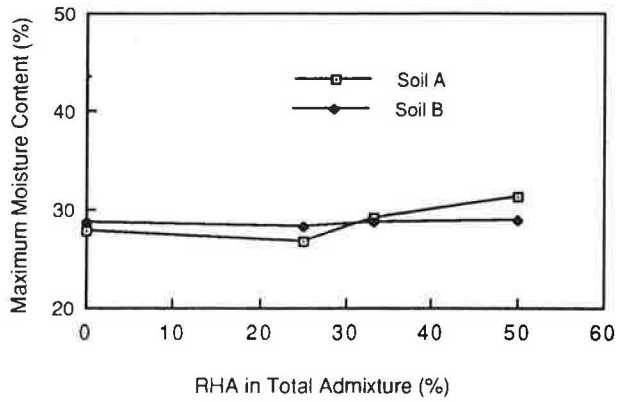

FIGURE 5 Maximum moisture content in cement–RHA-stabilized soils during wet cycles of wet-dry test.

9 for Soils A and B, respectively. The data points represent the average of two test results, and the coefficient of variation is less than 10 percent. Increasing cement content and curing period increases the strength of the treated silty soils. Table 3 and Figures 10 and 11, respectively, show the ratio of unconfined compressive strength of cement-stabilized (UCc) Soil A and Soil B to that of untreated soils (UC) at 7, 14, and 28 days.

For Soil A, the addition of 2 percent cement produces a strength ratio of 3.42 for a 7-day curing period and 11.14 for 28 days. For Soil B, corresponding values are 5.16 and 11.43. For 10 percent cement content, the corresponding strength ratios are 11.48 for 7 days and 18.65 for 28 days for Soil A; for Soil B, these values are 11.75 and 17.13, respectively. Clearly, increasing cement content five times does not produce a corresponding gain in strength for these soils. However, for both the soils, an appreciable increase in strength can be achieved by mixing a small amount of cement (i.e., 2 percent) and allowing curing.

At 10 percent cement content, the 7-day unconfined compressive strengths are 110.96 and 131.96 pounds per square inch (psi) for Soil A and Soil B, respectively. PCA (*12*) recommends a minimum 7-day unconfined compressive strength of 250 psi for soil-cement mixtures not containing material on a No. 4 sieve, 50 percent of which are smaller than 0.05 mm. The silty soils in the present study failed to achieve the specified strength at 10 percent content. However, both the soils have more than 50 percent materials smaller than 0.05 mm. Thus, the PCA recommendation regarding strength is of questionable validity when applied to typical silty soils in Bangladesh.

In the United States, the desired cement content is normally selected for durability. The implied assumption is that strength needs will automatically be met (*15*). This is not true for the silty soils evaluated in this research. The results in this study confirm the finding by Ahmed (*6*), who showed that stabilization of Bangladeshi silty soil of Type A4 requires 14 percent

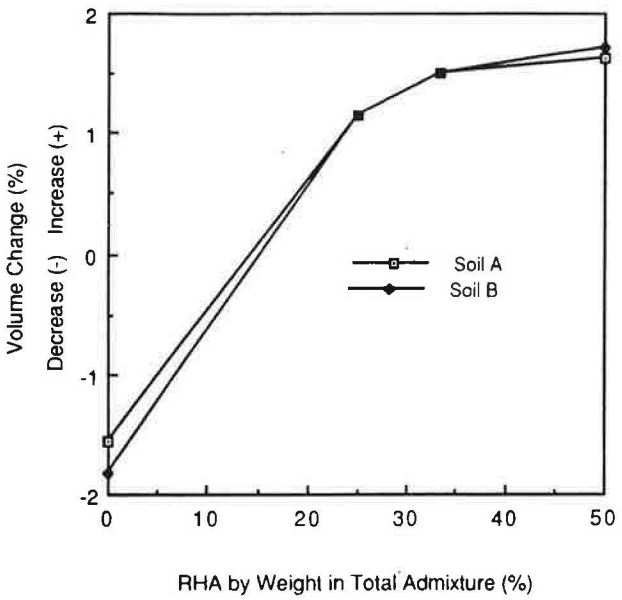

FIGURE 7 Volume change of cement-treated soils on addition of RHA in wet-dry test.

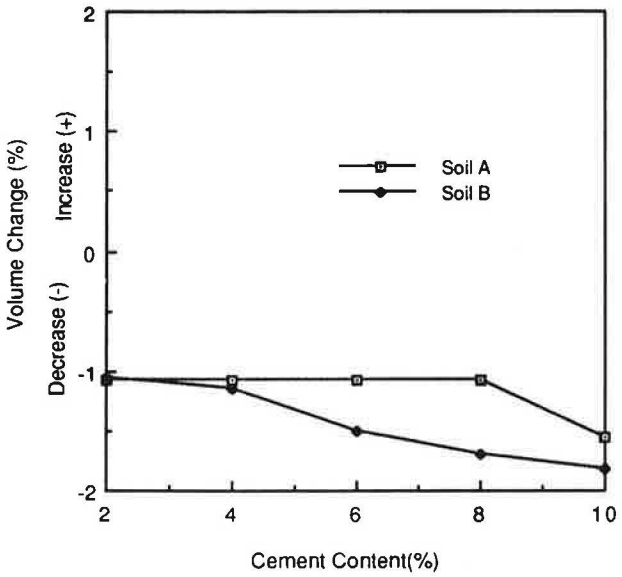

FIGURE 6 Volume change of cement-treated soils in wet-dry test.

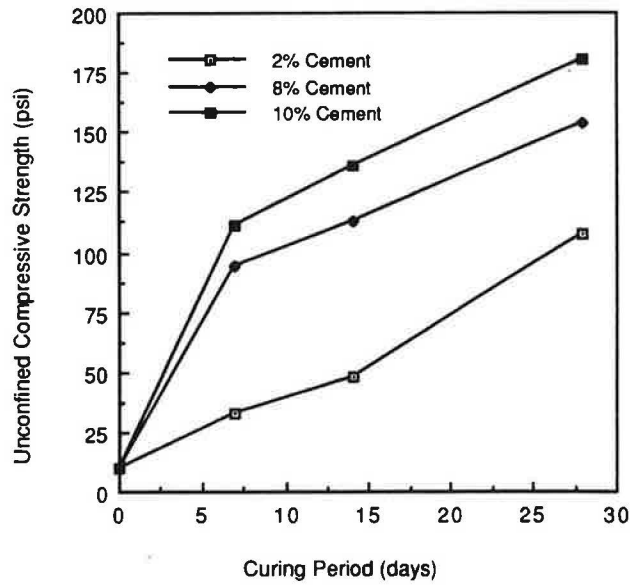

FIGURE 8 Effect of admixture content and curing period on compressive strength of Soil A.

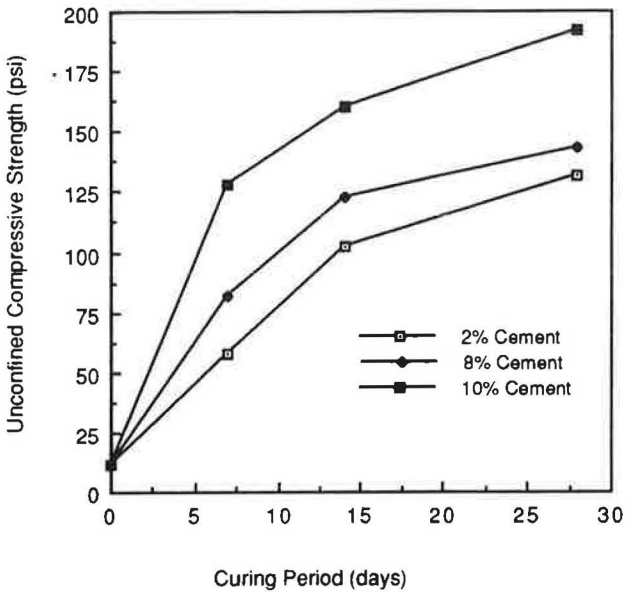

FIGURE 9 Effect of admixture content and curing period on compressive strength of Soil B.

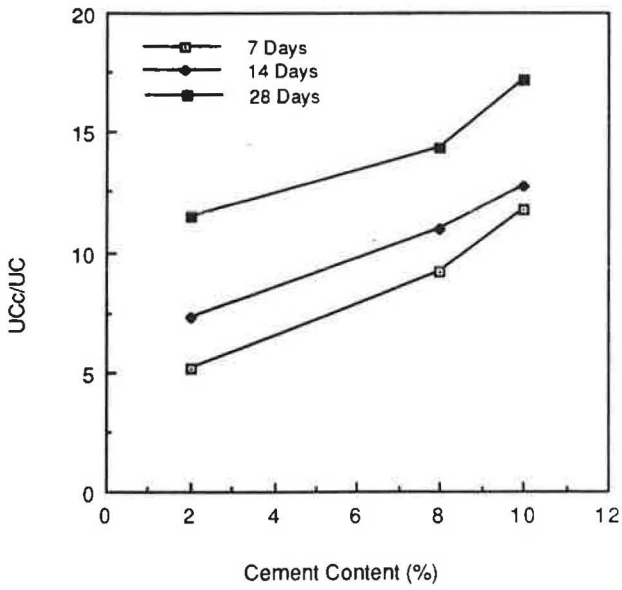

FIGURE 10 Strength ratio of cement-treated Soil A to untreated soil.

cement content and that Type A4 (*10*) requires a much higher cement content to meet the PCA strength criteria.

Effect of RHA on Strength

The effect of replacement of cement by RHA on unconfined compressive strength of Soils A and B is shown in Figures 12 and 13, respectively. Increasing ash content decreases strength relative only to cement-stabilized soil.

Table 4 and Figures 14 and 15 show the ratio of unconfined compression ($UC_{RHA\text{-}c}$) of stabilized Soils A and B using RHA and cement-blended admixture to that of cement-admixed soil.

For Soil A, 25 percent by weight of cement can be replaced by RHA with only a 5.5 percent decrease in 7-day unconfined compressive strength and with no change in 14-day unconfined compressive strength. The test results show that change is pronounced for ash contents higher than 25 percent and curing periods longer than 14 days.

For Soil B, lower 7-day compressive strength is obtained on addition of RHA. However, for longer curing periods and at 25 percent ash content, the results are similar. Hence, it can be assumed that, for alluvial silty soils of the A4 group, replacing 25 percent of cement with ash and a 14-day curing period produces strength almost the same as that produced by cement only.

Maximum Dry Density

The relationship between maximum dry density (AASHTO T99) and cement content is shown in Figure 16. For Soil B, density decreases with increasing cement content; for Soil A, density decreases up to 4 percent cement content, after which no change occurs.

Figure 17 shows the effect of addition of RHA on maximum dry densities of soil-cement mix for Soils A and B. Density decreases with increasing ash content. Because RHA has a

TABLE 3 RATIO OF UNCONFINED COMPRESSIVE STRENGTH OF CEMENT-STABILIZED SOIL (UCc) TO UNTREATED SOIL (UC)

Soil Sample	Cement Content(%)	UCc/UC 7 days	14 days	28 days
A	2	3.42	4.92	11.14
	8	9.71	11.73	15.90
	10	11.48	14.10	18.65
B	2	5.16	7.32	11.43
	8	9.14	10.93	14.28
	10	11.75	12.70	17.13

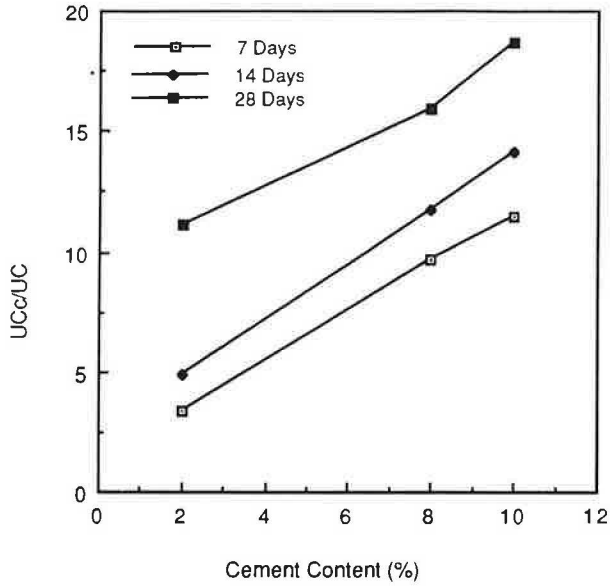

FIGURE 11 Strength ratio of cement-treated Soil B to untreated soil.

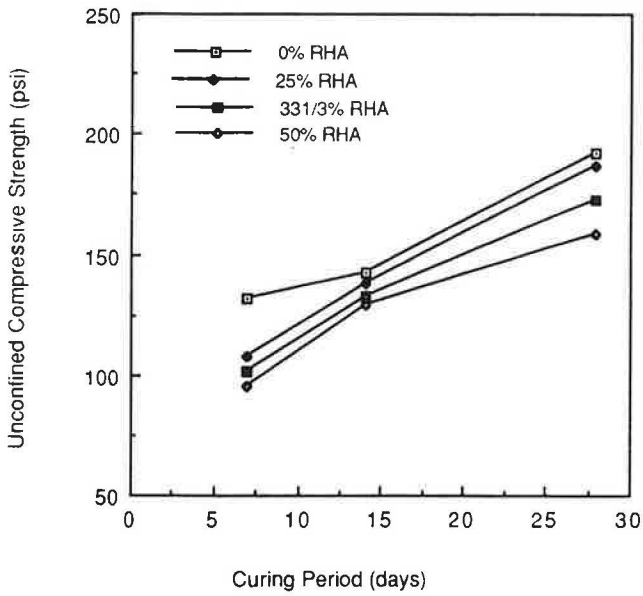

FIGURE 13 Effect of RHA replacement in cement on compressive strength of cement–RHA-treated Soil B.

unit weight less than the soil, the presence of RHA in the soil-cement mix reduces the density.

Plasticity Indexes

The variation of the Atterberg limits and the plasticity index with the increments of cement contents is shown in Figures 18 and 19, respectively. The plastic limit and liquid limit increase with increasing cement content. The increase in plastic limit is appreciable, lowering the plasticity index at higher cement contents. These results agree with those reported by Ahmed (*6*). Figure 20 shows that addition of RHA initially decreases the liquid limit and plastic limit of cement–RHA-stabilized soil. However, both increase at ash content greater than 33⅓ percent. This change is pronounced for the liquid limit and results in an increase of the plasticity index (see Figure 21).

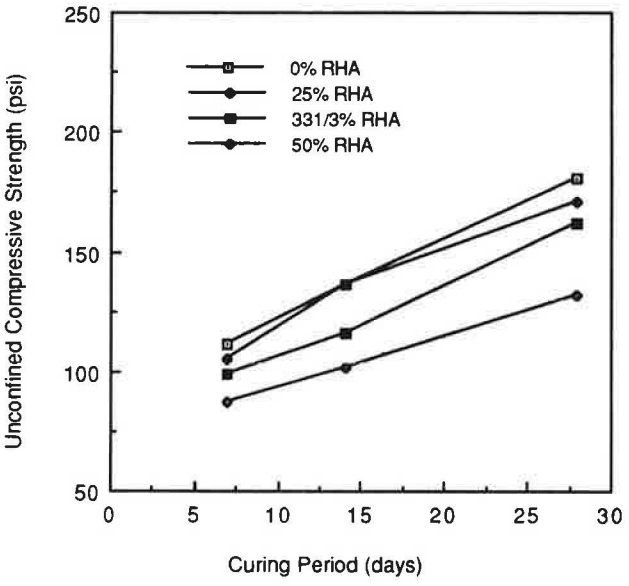

FIGURE 12 Effect of RHA replacement in cement on compressive strength of cement–RHA-treated Soil A.

Properties of RHA-Stabilized Soil

An attempt was made to study the properties of soils stabilized only with RHA. Samples prepared by mixing different percentages of RHA to soil for unconfined compression and plasticity tests collapsed during curing. Results indicate that, like many types of pulverized fuel ash, RHA has no cementation value of its own.

CEMENT AND CEMENT-RHA STABILIZATION OF LOW-VOLUME ROADS

Local alluvial silty soils require a very high amount of cement to fulfill the PCA strength requirement for soil-cement stabilization. In Bangladesh, where domestic cement production does not meet the local demand and cement is costly, the soil-cement stabilization technique is uneconomical with such a large amount of admixture. However, the stabilization of alluvial silty soils with a small amount of cement (i.e., 2 percent) is promising. The addition of 2 percent cement produces an appreciable strength increase. This cement-modified soil can be used as an upgraded material for subgrade and subbase construction on low-volume paved roads in Bangladesh. A similar study in Singapore (*16*) has demonstrated that silty soils with 2 percent cement can be employed successfully for good subbase or subgrade construction, satisfying TRRL requirements.

RHA can be used with cement to achieve economy in the construction of low-volume roads. This study shows 25 percent of the cement can be replaced by RHA for stabilization. If a minimum 6 percent cement content is used for stabili-

TABLE 4 RATIO OF UNCONFINED COMPRESSIVE STRENGTH OF CEMENT-RHA-STABILIZED SOIL (UC_{RHA-c}) TO CEMENT-STABILIZED SOIL (UCc)

Soil	Cement:	UC_{RHA-c}/UCc		
Sample	RHA	7 days	14 days	28 days
A	3:1	0.945	1.0	0.948
	2:1	0.890	0.853	0.897
	1:1	0.789	0.746	0.730
B	3:1	0.820	0.960	0.973
	2:1	0.770	0.927	0.897
	1:1	0.720	0.904	0.823

Note: Total Admixture Content is 10%

zation, then the cement requirement with partial replacement by RHA would translate to 4.5 percent cement content.

Considerable savings can also be achieved by treating the local soils with an RHA-cement blend for earth road construction. The roads are expected to be more durable than those constructed with untreated soil, reducing annual maintenance cost and providing a stabilized base or subbase for stage construction of paved roads.

CONCLUSIONS

The important findings and conclusions drawn on the various aspects of this study can be summarized as follows:

1. The alluvial silty soils satisfy the durability criteria recommended by PCA at about 9 percent cement content. This value is well within the suggested range.
2. The silty soils fail to satisfy the minimum unconfined compressive strength criteria for the cement content at which the durability criteria is satisfied. Thus, the results do not support the implied assumption that the strength needs will automatically be met if the durability needs are satisfied. For the type of soils used in this study, a much higher cement content would be required to satisfy the strength criteria.
3. RHA can be blended with cement to stabilize silty soils. The results suggest that a total admixture content of 10 percent can be replaced with 7.5 percent cement and 2.5 percent RHA without impairing the durability or appreciably decreas-

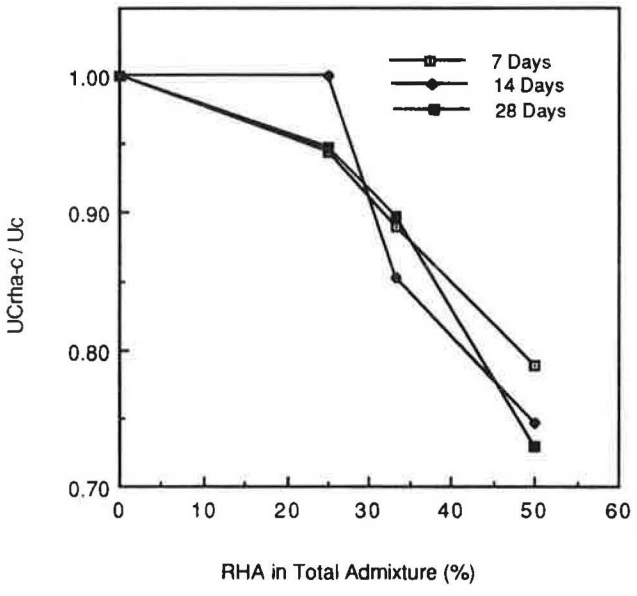

FIGURE 14 Strength ratio of RHA–cement-treated soil to cement-treated soil versus RHA content in total admixture for Soil A.

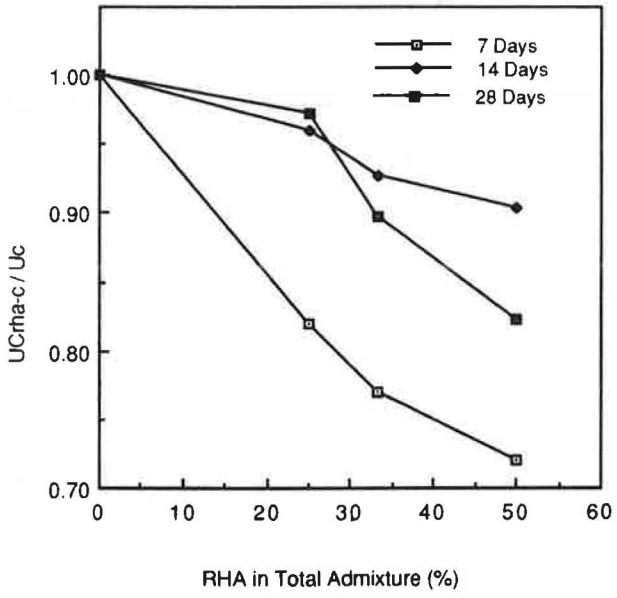

FIGURE 15 Strength ratio of RHA–cement-treated soil to cement-treated soil versus RHA content in total admixture for Soil B.

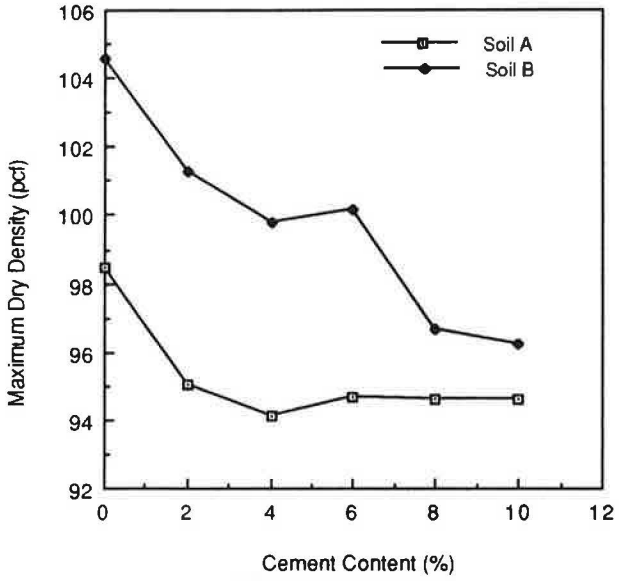

FIGURE 16 Effect of cement addition on dry densities of Soils A and B.

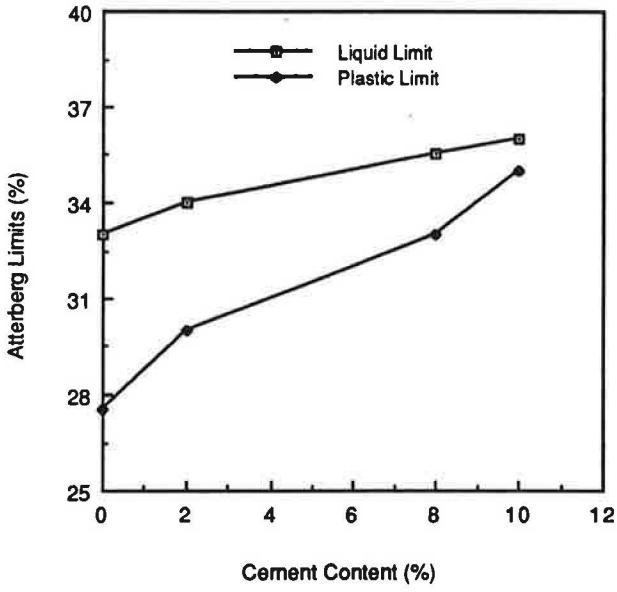

FIGURE 18 Effect of cement addition on Atterberg limits of Soil B.

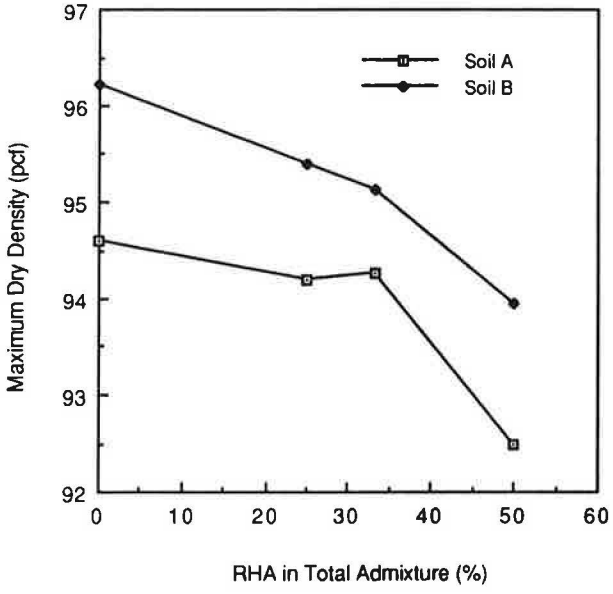

FIGURE 17 Effect of RHA addition on dry densities of cement-treated Soils A and B.

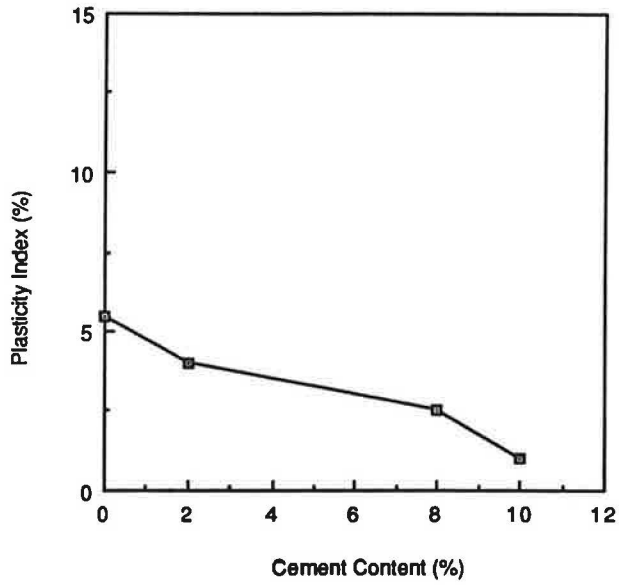

FIGURE 19 Effect of cement addition on plasticity index of Soil B.

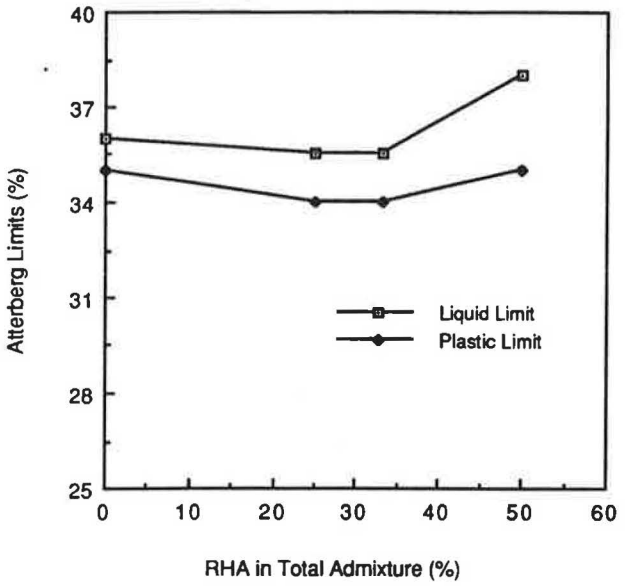

FIGURE 20 Effect of RHA-cement addition on Atterberg limits of Soil B.

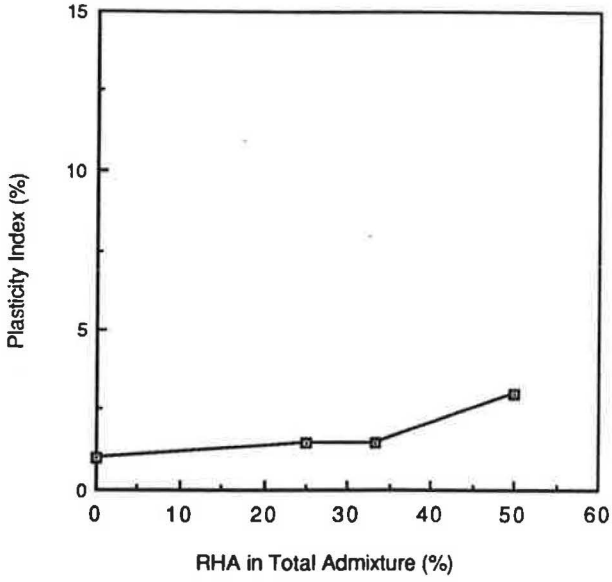

FIGURE 21 Effect of RHA-cement addition on plasticity index of Soil B.

ing the strength of such mixtures in comparison to those in which only cement is used.

4. A reduction in volume takes place during dry cycles of the wet-dry test with higher proportions of cement, whereas the addition of RHA results in an increase in volume on wetting. Silts show a decrease in maximum dry density when treated with cement. Cement–RHA-treated silty soils show a further decrease in dry density.

5. RHA cannot be used alone for stabilization of soil because of its lack of cementitious property. It can only be used as an admixture with other cementitious materials.

6. The soils showed an appreciable strength gain over untreated soil with addition of only 2 percent cement by weight. Silty soils treated with 2 percent cement can be used as a good subbase and subgrade material in low-volume paved-road construction. Earth roads stabilized with a cement-RHA blend are expected to be more durable than untreated earth roads, reducing annual maintenance costs and providing a good subbase or base for stage construction of paved roads.

ACKNOWLEDGMENTS

The authors wish to express their gratitude to the Department of Civil Engineering, Bangladesh University of Engineering and Technology, for financial support. Special thanks to A. Habib and J. R. Chowdhury for their personal interest in this study.

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Nonstandard Stabilization of Aggregate Road Surfaces

DOUGLAS E. SCHOLEN AND SKIP COGHLAN

A study of the performance of aggregate road surfaces treated with nonstandard stabilizers is described. Over 100 mi of road surfacing in a broad range of environments was treated with three general types of stabilizers: (a) pozzolans, (b) bioenzymes, and (c) an electrolyte. These stabilizers are byproducts of other industries and are generally unknown to road builders. When used appropriately, they have been found to be more effective than standard stabilizers. Factors considered in the evaluation included the aggregate gradation, types of failures reduced or eliminated, improvement in surfacing resilience, and economic benefits. The cost of application of these stabilizers was found to be comparable to the cost of aggregate replacement. The stabilization effected is permanent and long lasting. The benefits include substantial reductions in aggregate loss, reduced maintenance frequency, and improved road serviceability. Sources, recommended application percentages, and methods of application and mixing are described for each stabilizer. Several test or demonstration road sections are also discussed for each. Data on Clegg Impact Values obtained from these roads are presented. It is concluded that substantial economic and service benefits are to be realized from the use of these stabilizers.

During the past decade, a number of relatively unknown soil and aggregate stabilizers have been made available to the construction industry. Most of these materials are byproducts of unrelated processes, produced or modified specifically for use as stabilizers. Unlike traditional stabilizers such as lime, Portland cement, and bitumens, there are no standard laboratory tests in use to effectively predict the performance of these stabilizers in the field. Because their producers are generally unfamiliar with the construction industry, effective communication with builders has been lacking. The considerable benefits of the stabilizers remain undiscovered to large segments of the construction industry, many of whom are seeking effective solutions to longstanding problems.

An attempt was made to bridge this gap in communication by identifying some of the advantages and limitations of these products, and thereby help builders reap the benefits of the years of research that have gone into the development of these materials for use as stabilizers. Efforts were concentrated on three types of materials: (a) pozzolans, (b) bioenzymes (biocatalysts), and (c) an electrolyte. Brief descriptions of an acrylic polymer and a pine tar derivative are also included.

Because standard laboratory tests were not available, construction of test and demonstration road sections was primarily used. During the past few years, well over 100 mi of road surfacing or subgrade have been stabilized with these materials at sites scattered across the United States. The majority of these sites are on forest development roads (FDRs) on National Forest land of the U.S. Department of Agriculture (USDA) Forest Service, Southern Region. The prolonged and often intense rainfall of this area, together with the thousands of miles of highly erosive aggregate surfacing and vast exposures of expansive clay subgrade soils, had provoked an early interest in low-cost stabilizers among National Forest road managers. Other users of these materials include several state, county, and local government organizations, as well as private enterprise.

Region 8 of the Forest Service began trial installations of these materials in 1982. Initial efforts were based on reports from New Zealand (*1*) on their experience with modifying soils and aggregates using low percentages of Portland cement or lime. Attention shifted to other stabilizers as they became known. Project specifications have been developed for treatment of subgrades, aggregate bases, and aggregate surfacing. In 1987, the Federal Lands Highways Coordinated Technology Implementation Program (CTIP) approved a project to evaluate and report on these materials. The final CTIP report is scheduled for February 1992 and will include additional products and projects. This progress assessment includes evaluation of sections currently completed.

EVALUATION FACTORS

In evaluating the performance of nonstandard stabilizers, several factors dominated the concerns and benefits under consideration. These factors include aggregate gradation, surfacing resilience, and economy.

Aggregate Gradation

Aggregate gradation was noted to be an important factor in the performance of the treated aggregate surfaces, regardless of the type of stabilizer used (see Figure 1). The best performance was obtained from aggregates with 30 to 50 percent retained on the No. 4 sieve, and within this range the better graded aggregates showed the least surface damage under prolonged use without maintenance. These surfaces developed a well-armored appearance under traffic, similar to a bituminous surface treatment. Aggregates with less than 20 percent retained on the No. 4 sieve developed excessive fines on the surface and developed shallow ruts. Aggregates with more than 50 percent retained on the No. 4 sieve developed

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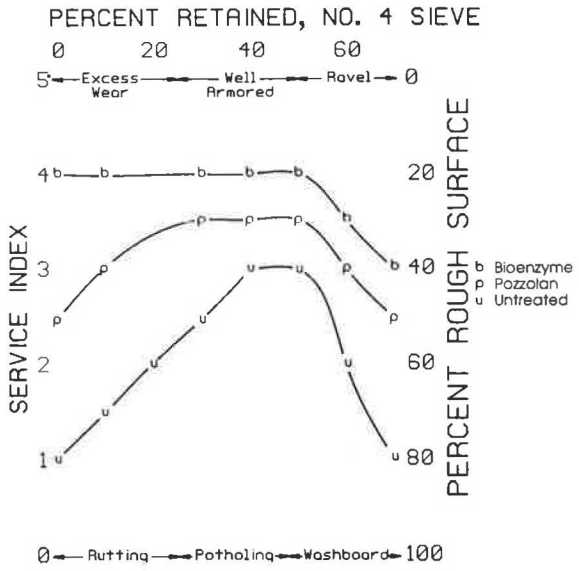

FIGURE 1 Performance of treated surfaces as related to percentage of coarse material in aggregate.

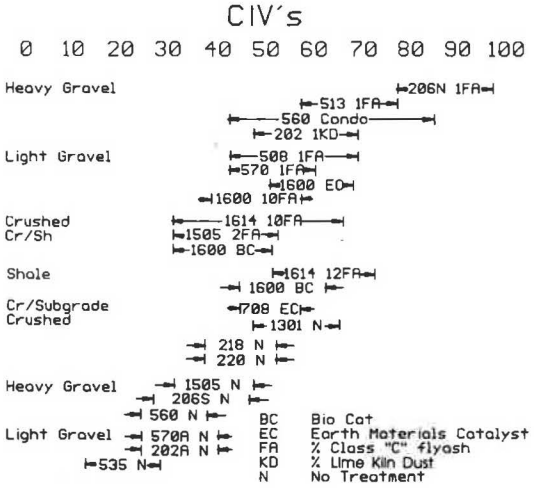

FIGURE 2 CIVs for untreated and treated aggregate surfaces.

excessive surface ravel in large aggregate, which has an abrasive effect on the surface under traffic. As this layer of loose material thickens, corrugations occasionally develop on grades.

Surfacing Resilience

The absence of standard testing procedures for evaluating the performance of these stabilizers leaves much to be desired in developing an objective report. However, the Clegg Impact Hammer has provided some small measure of improvement in surface resilience following treatment. The 10-lb hammer imparts only a small impact to the aggregate mass, enough to loosen an unbound aggregate but not enough to differentiate between a clay binder and the more effective stabilizer binder. Therefore, the Clegg Impact Values (CIVs) obtained show the greatest benefit for treatment of nonplastic aggregates, and generally no improvement for those aggregates with plasticity. Figure 2 shows a definite trend of increasing CIVs from the tests on untreated aggregates to those on treated aggregates.

Economy

The short- and long-term cost benefits are of paramount importance in evaluating a stabilizer. Surface aggregate replacement is the most costly item in maintaining aggregate surfaced roads (see Figure 3). The cost of blading is related less to grader operation than to the influence of blading on surface degradation and increasing the rate of aggregate loss. To provide long-term benefits, an effective stabilizer locks the aggregate particles in place and indefinitely maintains the original compacted density achieved during construction, preventing or substantially reducing aggregate loss and reducing or eliminating the damaging effects of frequent blading (see Figure 4). In the short term, the cost of initial stabilization should be in the same cost range as a single aggregate replacement operation to avoid unwanted increases in already strained budgets (see Figure 5).

Another long-term benefit is reduced wear on user vehicles, resulting in substantial savings to haulers, school buses, local residents, and recreationists. User satisfaction promotes a greater willingness to pay for additional treatment.

POZZOLANS

The benefits offered by pozzolan treatment include reduced aggregate loss and improved serviceability, with maintenance reduced to one or two light bladings per year on roads carrying average daily traffic (ADT) of 50 to 400, including logging trucks and oil well maintenance vehicles. At least half of the test sections, in Arkansas and North Carolina, are subjected to freeze-thaw conditions during 1 or 2 months each year. Investigators (*2*) report that lime-stabilized soils lose an estimated 10 to 12 percent strength with each cycle but regain strength during the hot months.

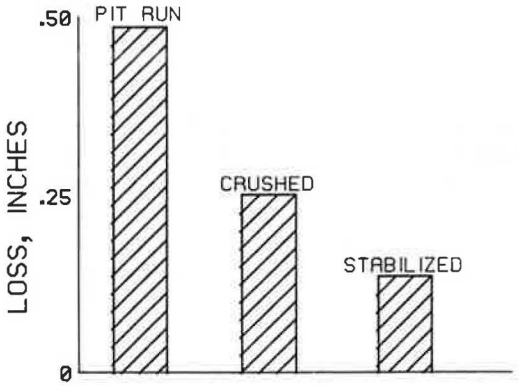

FIGURE 3 Comparison of aggregate loss for treated aggregate, untreated crushed aggregate, and pit run aggregate surfaces.

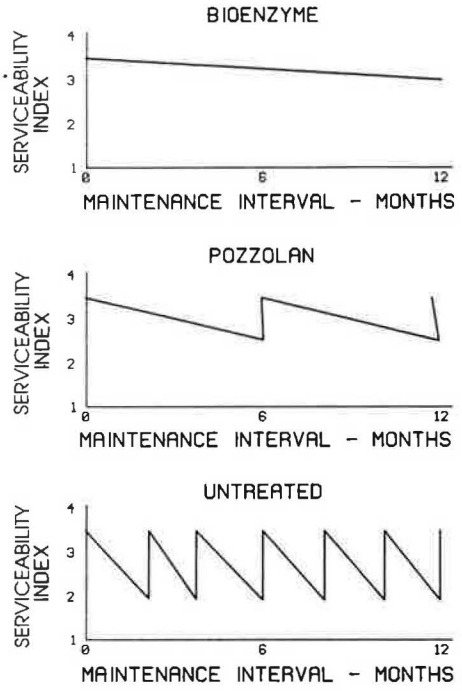

FIGURE 4 Serviceability index comparison for treated and untreated aggregate surfaces.

The pozzolans currently being tested include lime kiln dust, Class C fly ash, hydrated lime with Class F fly ash, and cement kiln dust. These materials are similar in that they are all hydraulic cements. Although they are waste materials, the distributors provide for periodic chemical analyses and make the results available to prospective users. Effects of variations in chemistry have not been noted in the field, but some differences might be observed in laboratory test results when smaller quantities are used. The pozzolans differ from Portland cement in not having a quick set. Strength gain is due to hydration and develops uniformly from initial compaction. The pozzolans are very effective in small percentages with nonplastic, coarse-grained aggregates and coarse sandy gravels. These treated aggregates can be scarified or rebladed at any time with moisture present and will resume strength gain upon recompaction. Silts and fine sands require substantially more additive, resulting in problems with mixing and development of slabs of cemented material in the roadbed that complicate maintenance. Pozzolan-treated aggregates with plasticity become extremely slippery during wet weather and must be covered with a traction course of crushed rock or washed gravel; the bioenzymes are a far more effective stabilizer for these aggregates.

The percentage of additive used in the test sections has varied from 0.5 to 2 for lime kiln dust and from 1 to 10 for Class C fly ash. Six percent cement kiln dust has been used in several projects, and 1 and 2 percent mixes of lime and Class F fly ash in a 1:1 ratio have been used on two 2-mi sections. The evaluation periods for these projects have ranged from a few months to 7 years. On the basis of observations to date, the percentage of additive required for satisfactory performance of a coarse aggregate is estimated to vary with the road grade from 1 percent for grades less than 2 percent to 3.5 percent kiln dust or 7 percent Class C fly ash for grades over 7 percent (see Figure 6). Using lower percentages than these resulted in development of surface corrugations. Occasional shallow potholing develops on flat graded sections due to washing of fines in puddles under traffic. A light blading once or twice each year provides for optimum service.

Kiln dust and Class C fly ash are both applied dry by pneumatic pumping through a spreader bar on the 24-ton tanker or by spreading from a dump truck with the tailgate cracked open. The road surface is shaped and scarified, and wet down thoroughly to blot the powder. In extremely windy areas, a slurry application is sometimes necessary. Mixing can be accomplished using scarifiers, rippers, chisel plows, or rotary mixers. To achieve optimum performance, 3 to 5 percent extra water is allowed for hydration of the quicklime fraction in the additive. Compaction with pneumatic or vibratory roller is accomplished to 95 percent of Standard Proctor or better and

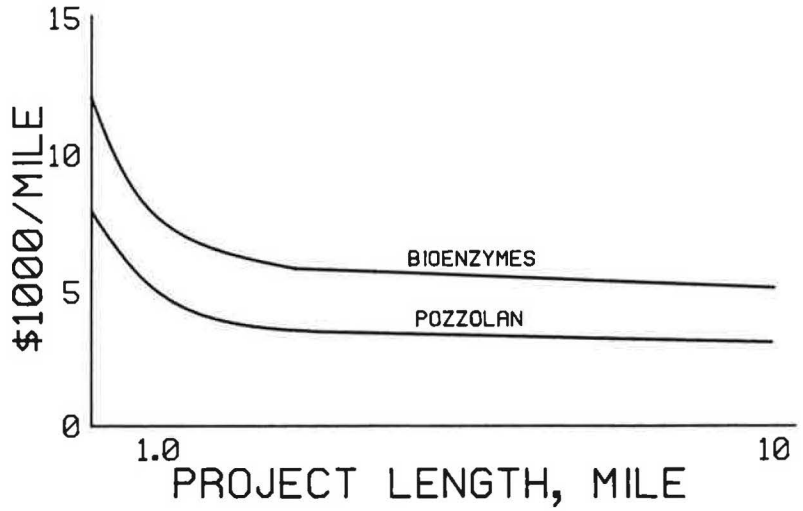

FIGURE 5 1988 construction costs for aggregate surface treatment with nonstandard stabilizers.

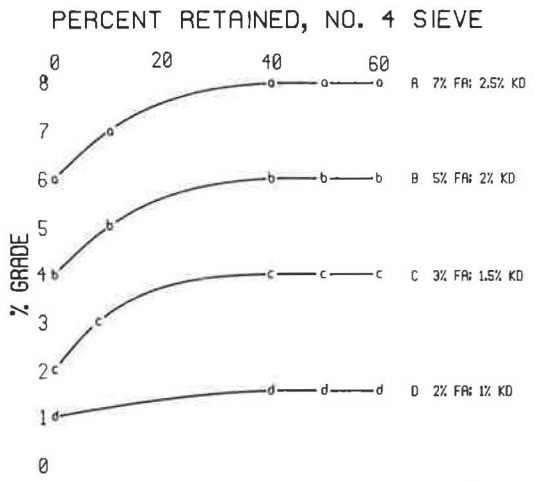

FIGURE 6 Estimated minimum percentages of Class C fly ash and kiln dust required to prevent corrugations on grades.

is followed by final shaping. Construction costs range from $3,500 to $7,000 per mile (1988 dollars), varying inversely with the project length (see Figure 3). Work production averages about 1 mi per day.

Lime Kiln Dust

Lime kiln dust is a byproduct of the lime manufacturing process. Crushed limestone and coal are fired together in a revolving kiln at high temperatures to produce quicklime. During this process, a waste dust is drawn from the kiln. The dust contains a percentage of fines similar to that of Portland cement and contains 35 to 45 percent quicklime, with the remainder being coal ash. When mixed with water, the dust hydrates, reacts, and hardens to form a cemented mass. It must be provided with dry storage and cooled for use as a stabilizer. Pozzalime is a lime kiln dust distributed by Mineral By-Products, Inc., in Marietta, Georgia. Soildry is a lime kiln dust distributed by Industrial Rock Mineral By-Products Company in Birmingham, Alabama.

Projects using lime kiln dust are located in Alabama, Virginia, Tennessee, North Carolina, Mississippi, and Florida, and all were constructed by the Forest Service.

National Forests in North Carolina

In 1985, North Carolina agreed to construct a test section on Davidson River Road using a very small percentage of Pozzalime with crushed rock containing about 25 percent fines. Twenty-four tons of Pozzalime were spread over 2 mi of 18-ft road, disked in to a 4-in. depth, watered, and compacted to 95 percent of AASHTO T99 density. This process amounted to about 0.6 percent additive overall, and 2.4 percent of fines only.

Blading on this road was reduced from nine to two times per year following this construction, and the surface performed satisfactorily during the 18-month period following construction. Grader operators noted substantial hardening on the treated sections during 1986. The winter of 1986–1987 was unusually warm, resulting in many freeze-thaw cycles (estimated at one per day). During a recent inspection, it was not possible to discern any difference in hardness between the treated and untreated sections.

On the basis of this performance and reports from Mississippi, the Forest Service elected to increase the Pozzalime application to 24 tons/mi on an additional 13.1 mi of gravel-surfaced road constructed during October and November of 1987 (Davidson River Road, 1.7 miles; Yellow Gap Road, 3.5 miles; Cathy's Creek Road, 1.4 miles; Headwaters Creek Road, 6.4 miles). The locations selected were primarily on grades where maximum gravel loss had been occurring.

The Pozzalime sections have been bladed once in the spring and once in the fall since construction. The Pozzalime has been effective in preventing separation of the coarse and fine fractions and in holding the shape of the section. However, blading was required in a few areas to repair shallow potholing on flat sections and corrugations on grades. The blading was accomplished with moisture present, and the bladed material knitted well to the roadbed, again providing performance superior to the untreated road sections.

National Forests in Mississippi

Construction of the FDR 202 test section, consisting of mixing 24 tons (1 percent) of Pozzalime into the top 4 in. of the first mile of road east of State Highway 15, was completed under contract in early October 1986. Because gravel replacement and renovation of 11 additional miles of FDR 202 were already planned for 1987, the Forest Service included Pozzalime treatment at the same rate in the project. The project was completed under contract in early September 1987. Most of the road has grades of less than 2 percent.

During an inspection in August 1990, the first mile of FDR 202, which had no gravel added with only 18 percent retained on the No. 4 sieve, now appears to lack coarse aggregate and had a full inch of loose silty sand showing wheel tracks and occasional shallow ruts. The section provides a smooth ride with slight fishtailing. The remainder of FDR 202 (11 mi) was treated with added gravel, and now has a thin float of sandy gravel over a hard cemented or packed aggregate with no corrugation, potholes, or ruts on the 8 mi west of FDR 206. It also has some slight corrugations east of FDR 206 due to sections of steeper grade or poor subgrade. The entire road provides a pleasant, easy ride at 40 mph with good control on smooth curves.

Class C Fly Ash

Class C fly ash is collected by electrostatic precipitators from the stacks of power plants burning Wyoming coal. This coal contains limestone, which converts to quicklime during combustion. Fly ash developed from this coal contains about 25 percent quicklime, considerably less than the 40 percent in the lime kiln dust.

Projects using Class C fly ash are located in Mississippi, Louisiana, Arkansas, and Texas, and all were constructed by Region 8 of the Forest Service. Class C fly ash is distributed

throughout the South by Ash Management Corporation of Georgia. Another local distributor is Flyash Products, Inc., in Little Rock, Arkansas.

Ozark National Forest in Arkansas

Sorghum Hollow Road, FDR 1614, had 1,000 ft of shale surfacing treated with 10 percent Class C fly ash in 1984. This section of heavily traveled road (80 to 100 ADT) has been bladed only twice since then and has shown little sign of wear, indicating that considerable hardening on the black thissle shale resulted from the treatment. On another 1.5-mi section of FDR 1614, a 2-in. open graded base aggregate (Arkansas Gradation SB-2) was mixed in equal quantities with the black shale and treated with 10 percent Class C fly ash in 1988. This section has not been bladed since and provides a smooth ride to the user.

New Blaine Road, FDR 1600, has a traffic count of around 200 ADT. On 2 mi of this road, a mixture of SB-2 and GB-3 was mixed with equal quantities of silty clay subgrade by scarification and treated with 10 percent Class C fly ash in 1988. These sections have been given a light blading twice per year and provide excellent service in all types of weather.

The successful use of 1 percent Pozzalime in Mississippi prompted interest in reducing Class C fly ash percentages in Arkansas. White Rock Road, FDR 1505, is a high-use road with grades up to 9 percent. On 5 mi of this road, the ¾-in. minus well-graded rock with clay fines was treated with 2, 5, and 10 percent Class C fly ash on separate sections 2 years ago. The road has been bladed once per year plus one time following a heavy hauling contract. Moderate corrugations have developed on all grades over 5 percent in the 2 percent fly ash section. No corrugations have been noted in the 5 percent fly ash section, which includes grades of 6 percent; in the 10 percent fly ash section, which includes grades of 9 percent; or in the 2 percent fly ash section on grades of 5 percent or less. A uniform, thin float of coarse aggregate was noted on all sections.

These sections have provided particularly useful insights to the percentages of additives required to prevent corrugations on steeper grades (see Figure 6).

National Forests in Mississippi

Two roads on poor subgrades had aggregate added and were treated with 1 percent Class C fly ash in June 1989. Both rutted during the winter months due to saturation of the expansive clay subgrade. This winter was the worst in recorded history, with double the normal rainfall. Both of these roads have now been recommended for subgrade treatment with Condor SS (described in a later section). Grades on these roads are less than 2 percent. FDR 513, which is 2.5 mi long, currently has a hard cemented appearance, with coarse aggregate well locked up in the fines. There are no corrugations or chuckholes, and wheel tracks are dished out from slight to occasionally heavy, indicating subgrade deformation. There is a thin float on the shoulders but no raveling, and the road provides a smooth, easy ride. FDR 508, 14.8 mi long, was recently bladed and has a uniform, hard surface without corrugations, rutting, or potholes.

FDR 206 North, located mainly on a good subgrade, had aggregate added and was treated with 1 percent Class C fly ash in June 1989. This section fared well through the winter and has an extremely hard, cemented, well-gravelled surface with minor float and no potholes or rutting. There is only an occasional minor washboard, and the road provides easy driving at 40 mph. It carries heavy oil well maintenance traffic year-round.

FDR 206 South, surfaced with the same aggregate as the northern section, has not been treated and exhibits extremely heavy corrugations, especially on the curves. This section is difficult to drive, and loss of steering control occurs at 35 mph.

Kisatchie National Forest in Louisiana

During September 1988, 4.7 mi of surfacing was completed using Class C fly ash, with a 24-ton load on each ¾ mi. Before adding the fly ash, portions of the road aggregate were thickened with a pit run sand and gravel mix of plasticity index (PI) 17. This unfortunate selection of material resulted from a lack of good aggregate sources in the immediate area. The remainder of the road had the original nonplastic sand and gravel surfacing. Following completion of construction, heavy rains caused slippery conditions on the high PI aggregate on grades, and a traction course of gravel was added on steeper grades. On sections of high PI aggregate that did not receive a traction course, the surface was softened by the unusually heavy winter rains, and 1- to 2-in. ruts developed. No problems were encountered on the nonplastic sections, which required no maintenance. This experience emphasized the importance of careful coordination in selecting aggregates and construction procedures.

Future Studies on Fly Ash Use

At the Department of Energy in Laramie, Wyoming, plans were developed during 1989 for implementing a $350,000 project to determine the chemistry and mechanics behind the successful use of Class C fly ash. Facilities at the Western Research Institute and the University of Wyoming were selected for the project. These facilities include an electron microscope, X-ray and spectrographic analyses, a full-scale climate and hydrology simulation laboratory, and geotechnical engineering testing capabilities. Concurrent with a 1-year laboratory study to determine optimum percentages of fly ash and to develop standard testing procedures and specifications, test strips will be constructed by state, county, and Forest Service organizations in Wyoming. These will be monitored over a 3-year period, after which the technology will be extended statewide and to other areas as appropriate. One of the goals will be to develop a standard laboratory procedure that can predict performance of fly ash–treated aggregates.

BIOENZYMES

Bioenzyme soil and aggregate stabilizers were developed as a byproduct of the enzyme industry, which specializes in food

processing, cleaning agents, and cosmetics. Industry personnel have only vague notions about procedures used in earth construction. Effective communication with the construction industry has been achieved only through the efforts of the better-trained product distributors.

The mechanisms of bioenzyme stabilizers are proprietary and secret. However, their general nature is understood by biochemists and is alluded to in advertising material. Bioenzyme stabilizers provide a bacterial culture in an enzyme solution. When exposed to the carbon dioxide in the air, the bacteria multiply rapidly and produce large organic molecules, which the enzyme attaches to the clay molecules in the aggregate, blanketing ion exchange points in the clay. This action prevents further absorption of moisture and results in a stable construction material. During the hydration that follows compaction, ionized water forms linkages between the closely packed particles, providing the cementing bond. The stabilizing effect of organic ions on clays has been discussed by Grim (*3*).

The observations made during and after construction on several projects support the premise that an ion exchange takes place between the alkali ions in the clay lattice and the organic ions provided by the biochemistry of the stabilizer solution. During mixing on the project in Montana, with ample clay present, clay lumps were noted to break down rapidly and lose plasticity. On the Montana project, and those in Texas and South Carolina that had clay present, the road surface hardened noticeably after 24 hr, indicating that hydration was causing cementation of adjacent sand grains. Full strength is reached well within the 4 or 5 days curing time recommended by the manufacturers (see Figure 7).

With clean crushed basalt in Washington and Oregon, where the only fines were rock dust, no reaction was observed and no reduction in maintenance was reported. Some clay content is essential to a successful bioenzyme project; a well-graded mix provides the best performance.

Bioenzymes are applied at the rate of 1 gal of bioenzyme per 9 to 15 yd^3 of aggregate. The bioenzyme is added to the compaction water and applied from the water truck in raising the aggregate moisture content to optimum. Wet aggregates must be dried back at least 2 points below optimum before application. Compaction is critical to good performance; the density will not change after compaction, and a poorly compacted surface can ravel. The highest density practically possible should be obtained. The road surface should be scarified before application to prevent runoff of solution. Mixing can be done with scarifiers, rippers, chisel plows, or rotary mixers, followed by compaction with a pneumatic or vibratory roller and a final shaping. About 5 days are required for a full cure, but light traffic need not be interrupted by construction. Construction costs in 1988 ranged from $6,000 to $12,000 per mile, varying inversely with project length (see Figure 3). Work production averages about 1 mi per day.

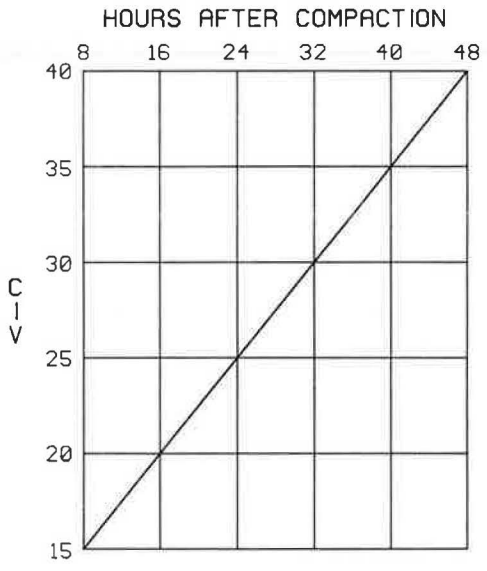

FIGURE 7 Rate of curing for bioenzyme stabilizers, expressed as a function of CIVs.

Test sections and trial roads using a variety of bioenzymes are located in several states. Four types are discussed in the following paragraphs.

BIO CAT 300-1

BIO CAT 300-1 is a bioenzyme manufactured in Nevada and distributed by the Soil Stabilization Products Company of Merced, California. The application rate is 1 gal/9 yd^3. Test sections are located in Nevada, North Carolina, Texas, Montana, Georgia, South Carolina, Michigan, Pennsylvania, and Arkansas.

Rain Mine Access Road in Nevada

The Newmont Gold Company's 13-mi access road to the Rain Mine south of Carlin, Nevada, was constructed in 1987 and surfaced that fall with an exceptionally well-graded pit run siltstone talus, containing 10 percent clay fines. The aggregate was stabilized with BIO CAT and surface crusted with magnesium chloride ($MgCl_2$). The 20- to 30-ft-wide, balanced, cut-and-fill road winds upslope at grades up to 10 percent to the ridgetop mine location. The late season construction limited surface thickness to 4 to 6 in. on the first 5 mi and 2 to 3 in. over the remaining 8 mi. The full thickness was completed a year later. Following compaction under grid rollers, the BIO CAT–treated material was covered with 0.6 to 0.75 g/yd^2 $MgCl_2$, which crusted on the surface with minimal penetration.

Throughout the winter months of 1987–1988, traffic included daily commuting of 500 construction personnel working at the site, as well as transport of equipment and materials for the rock crusher and ball-and-roller processing plants. The only maintenance required during this period was snow removal, which was accomplished using a grader blade to scrape the hard aggregate surface and applying a sand and salt mixture.

Depending on exposure, some sections were subjected to repeated freeze-thaw cycles, whereas others endured prolonged freezing and a spring thaw. The road surface remained free of rutting, potholing, and corrugations, although some surface ravel was noted on curves in the 2-in. sections. The subgrade over most of the road is of broken shale and siltstone, and in some areas bedrock can be seen in the surfacing.

Maintenance has consisted of roller brushing the surface at 2-week intervals, snow removal during the winter, and an

annual spring application of magnesium chloride. The completed surface gives the impression of a paved road rather than gravel.

Pisgah National Forest in North Carolina

During October 1987, BIO CAT was applied to a ½-mi section of Yellow Gap Road and mixed into the 4-in. crushed gneiss surfacing. This crushed rock contained about 15 percent fines, including a 2 percent clay fraction. The mixture had stiffened noticeably during reshaping on the second day and set up very hard after compaction. The performance of the section since construction has been exceptional. In all seasons, the road accommodates log hauling, hunting traffic, and other recreational traffic. Although untreated sections of this road are bladed three times annually and deteriorate rapidly after each blading, the BIO CAT section has required no blading other than ditch cleaning for over 2 years and essentially remains in its original condition. All surface gravel is firmly locked into the road surface, giving the effect of a surface treatment.

Sam Houston National Forest in Texas

In August 1989, BIO CAT was applied to 4.5 in. of aggregate surfacing on 2.5 mi of 20-ft double-lane road on FDR 204. This road has been in operation for many years, providing access for log hauling and to residential, day use, and lake areas. The entire road has an expansive clay subgrade. Before treatment, potholing often became so extensive that drivers resorted to using the road shoulders.

A mix of materials has been used during maintenance work, leaving at least five material types along the road. These included various combinations of poorly graded quartz gravels and crushed river rock, mixed with small amounts of sand, and red or brown clay binder (7 percent), crushed sandstone, and clay balls from the gumbo subgrade in one 200-ft section.

The surfacing hardened noticeably after construction and remained in excellent condition through the hunting season and heavy early winter. The unusually heavy winter rains resulted in subgrade failures, with minor rutting and shallow potholes developing. In March 1990, the surface was scarified, shaped, and recompacted at optimum moisture. Since then, it has developed a well-armored surface without potholes, corrugations, or rutting. Ravel is developing on some sections due to excessive amounts of 3- and 4-in. rock in the aggregate.

Lewis and Clarke National Forest in Montana

In September 1989, BIO CAT was applied to sections of Spring Creek Road. An inch of clay shale was added to the existing 4 in. of crushed sandstone surfacing over 32 mi of road to provide a binder, and the two materials were blended and compacted. BIO CAT was selected to stabilize 2.5 mi of this surfacing. The sections selected for application of the BIO CAT are on grades that had previously caused notable problems with corrugations. Mixing was accomplished using a combination of scarifiers and windrow blading, followed by compaction under a steel-wheel vibratory roller with two wide tires following. Except in shaded areas, the compacted surface was hard and uniform; after 24 hr, major portions had indurated and could not be scratched with a fingernail.

Treated sections have remained in excellent condition since construction, whereas untreated sections on gentler grades showed signs of deterioration in September 1990.

Chattahoochee National Forest in Georgia

In May 1990, a half-mile section of Tallulah River Road, FDR 70, had its poorly graded crushed-stone surface treated with BIO CAT. Before treatment, the surface was scarified into the subgrade to bring up plastic fines. The mix contained 11 percent passing a No. 200 sieve, and 1 percent clay. Mixing was accomplished with grader scarifiers. The entire section is on a flat grade and receives heavy traffic throughout the year. Soon after treatment, two 100-ft sections developed shallow potholes on the crown due to loss of fines in puddles. The potholes do not affect traffic and have not increased in size. The surface has a well-armored appearance, without ravel, and requires no maintenance.

Sumter National Forest in South Carolina

During July 1990, construction of 2.8 mi of bioenzyme-stabilized surface was completed on Burrell's Ford Road, FDR 708. The silty clay subgrade material was scarified into the existing poorly graded base rock, providing about 35 percent passing a No. 200 screen and including 2 percent clay. Several sections of the aggregate have excessive amounts of 2-in. coarse rock. The mixture was treated with BIO CAT on half of the road and with Earth Materials Catalyst (discussed in the following subsection) on the other half. Mixing was accomplished with an extra-heavy-duty chisel plow. Since construction, the surface has remained hard and smooth, except for a total of 600 ft on six sharp curves and a short, steep tangent. On these sections, some moderate raveling and corrugations have developed due to erosion rilling and excessive coarse rock in the aggregate. Traffic on this road, which has grades up to 9 percent, exceeds 90 veh/day during high-use periods.

Earth Materials Catalyst (EMC)

A bioenzyme known as Earth Materials Catalyst (EMC), also distributed by the Soil Stabilization Products Company, was applied to 1.0 mi of Logging Creek Road 968 and adjoining Lick Creek Road 67 in Lewis and Clarke National Forest during October 1989. It was also applied to half of Burrell's Ford Road in the Sumter National Forest as described in the previous subsection. A third test section was located in Arkansas. The performance of EMC has been comparable to that of BIO CAT. EMC is applied at the rate of 1 gal/12 yd^3, compared with 9 yd^3 for BIO CAT. EMC is manufactured in Arizona.

Permazyme

The bioenzyme Permazyme, also called Endurazyme, is distributed by American Enzymes of Alexandria, Virginia. Per-

mazyme is recommended for use with subgrade soils containing clay. In April 1989, the Kisatchie National Forest completed the FDR 321 spur—1,000 ft on native clay. Logging in this area is expected to start early in 1991.

The city of Raleigh, North Carolina, recently constructed a 600-ft test section in Lake County on a dirt street. The treated soil set up hard and is performing well. California bearing ratio (CBR) tests run on the material showed a substantial gain in strength after treatment.

Two city streets in Stillwater, Oklahoma, were surfaced with 6 in. of silty clay subgrade material (PI, 20; LL, 40) stabilized with Permazyme. No other surfacing was added other than an inch of gravel to improve traction and provide a slime-free surface. One of the streets, a half-mile of Richmond Road, was completed in 1981 and has not required maintenance since then. Adjacent untreated sections rut easily following rainfall and become impassible during spring thaw. The test section has been used year-round without problems or signs of rutting, chuckholing, or corrugations, remaining as hard and firm as the dry, baked clay of mid-summer. Because most of the gravel layer had been lost to traffic, the surface appears to be clay with imbedded gravel fines.

Another test section is located in the Peoples Republic of China.

PSCS 320

PSCS 320 is a bioenzyme distributed by Alpha Omega Enterprises of Richardson, Texas. It is recommended for use with expansive clay subgrades. A recent project was a 0.4-mi timber haul road constructed by Temple-Eastex Corporation on their timber lands north of Houston, Texas.

ELECTROLYTES

Electrolytes affect the basic nature of the clay molecules in the aggregate, causing them to release absorbed water and coagulate into a dense, moisture-free mass that resembles rock. For this stabilizer to be effective, aggregates must have 35 percent passing the No. 200 sieve with a clay fraction. Once stablized and compacted, they remain unaffected by wet-dry or freeze-thaw cycles. The electrolyte travels through native soil moisture by osmosis, and thus does not have to be mixed mechanically with the soil layer. The treatment is best applied during periods of soil saturation and is highly effective in permanently reducing the moisture content in expansive clays, eliminating subgrade and foundation problems associated with these troublesome soils. The stabilizer solution can be applied by injection for a deep treatment or by scarifying and flooding for a shallow treatment. The latter produces an extremely slippery condition and requires addition of a traction course if no coarse aggregate is present.

Condor SS is currently the only known electrolyte on the market that can be effectively used for soil stabilization. It is distributed by Earth Science Products Corporation of Wilsonville, Oregon, the Pro Chemical Stabilization Company of Dallas, Texas, and the Soil Stabilization Products Company of Merced, California.

Condor SS was used by the Forest Service on FDR 3421 spur in the Gifford Pinchot National Forest in Washington State to stabilize an impassible steep grade on a saturated clay loam soil. It was injected at 6-ft intervals to a 3-ft depth using a high-pressure ceramic pump and jetting pipes. Immediately following the treatment, heavy construction equipment was able to pass over the section without rutting.

The Oregon Department of Transportation injected Condor SS to stabilize a pumping subgrade on 2 mi of Oregon State Highway 47 between Mist and Vernonia (near Portland).

Injection of the electrolyte by the city of Pocatello, Idaho, has prevented severe frost heave from developing in certain city streets underlain by a moisture-sensitive loess soil, compared with adjacent control sections. Pocatello plans to apply the electrolyte on 3 mi of streets per year until all problem areas have been completed.

The Dow Chemical Company has stabilized 2 mi of causeway by injecting Condor SS near Galveston, Texas, to eliminate subsidence. Also in Texas, the city of Houston and Houston County recently ran tests to compare the Condor treatment with lime treatment on expansive clays and found Condor by far the more effective.

The Kisatchie National Forest stabilized 2 mi of expansive clay subgrade on FDR 560 near Winnfield, Louisiana, using the injection method. In a second project in the Kisatchie, a half-mile logging spur with an expansive clay subgrade was scarified and flooded with the solution, allowed to drain, and then compacted. It now remains brick hard through intense rainfall.

OTHER STABILIZERS

Several other stabilizers have been included in the study. Among these are Exxon Polybuilt 4178, Road Oyl, and materials traditionally used for dust control.

Polybuilt is an acrylic polymer used for erosion control on earth slopes and distributed by Exxon in Atlanta, Georgia. The Chattahochee National Forest used this material to prevent erosion on a roadside slump in a highly erosive granite saprolyte. The polymer was sprayed on with the grass seed, and effectively held the seed in place while the grass took root, despite several intense rainfalls. The slope across the 1-acre area ranged from 50 to 70 percent. Polybuilt can also be mixed with sand to form a pavement. A study test section has been planned but not yet constructed.

Road Oyl is a pine tar derivative manufactured near Knoxville, Tennessee, and distributed by the Soil Stabilization Products Company of Merced, California. Road Oyl can be substituted for asphaltic materials used in bituminous pavements and surface treatments and will achieve a Marshall stability of 4000 to 5000 lb, compared with 1,500 to 2,000 lb for asphalt. The product has been used primarily on mine access roads in the Knoxville area. Several study test sections are in the planning stage.

Dust palliatives such as magnesium chloride and lignin sulfonate provide temporary stability to road aggregates. Because they are water soluble, however, they must be reapplied periodically depending on rainfall frequency and intensity. Although light rainfall will improve performance, intensive rainfall with flooding rapidly leaches these materials from the road surface. Study test sections for lignin sulfonate have been constructed by FHWA and the Forest Service in the states of

Washington, Oregon, New Mexico, and North Carolina. Magnesium chloride is being tested by the Forest Service in Washington and Arkansas.

CONCLUSIONS

The final report for this study is due in February 1992, providing an additional winter for evaluation of the test sections. The performance through September 1990 shows an exceptional improvement over control sections wherever the appropriate stabilizer has been used. Failures in the test sections have been attributed to misuse of the stabilizer or poorly graded aggregates.

The observed improvements in performance and reductions in maintenance far exceed those of control sections or any other untreated aggregate surfaces. The bioenzymes and Condor SS have been particularly outstanding, in some cases extending maintenance frequencies from biweekly to biannually for similar performance. The low construction costs for these materials and for the pozzolans can easily be offset in reduced aggregate loss, reduced maintenance, and improved serviceability.

The primary drawback is the absence of standard testing procedures capable of predicting performance. The proposed laboratory study by the Department of Energy in Laramie, Wyoming, may provide some relief. Hopefully, other agencies, industry, and users will also contribute to developing the needed design and construction standards.

ACKNOWLEDGMENTS

Credit for the success of this effort must go to the many managers, engineers, and contractors having the interest and patience to implement a new technology on their construction projects. Funding support and assistance from the CTIP steering committee brought this project together.

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Permanent Road Stabilization: Low-Cost Pavement Structures and Lightweight Geotextiles

GEORGE A. CICOFF AND C. JOEL SPRAGUE

Greenville County, South Carolina, constructs permanent paved surfaces on approximately 20 mi of existing gravel roads each year. The county engineer sought to protect his new low-cost pavement from premature degradation and more frequent maintenance by including an appropriate geotextile as a separator between the subgrade and the pavement structure. The physical properties required to make the geotextile an effective long-term separator include both strength properties, which resist the forces of coarse aggregate being pushed into the subgrade, and hydraulic properties, which prevent the pumping of fine soils up into the coarse base aggregate while still allowing for pore water pressure dissipation from the subgrade. The geotextile strength and hydraulic properties necessary to survive construction and to provide long-term filtration and separation between the subgrade and the base aggregate are evaluated by testing of exhumed geotextile samples and visual pavement condition surveys for a trial installation.

The geotextile acts as an effective long-term separator by resisting the forces of coarse aggregate being pushed into the subgrade, and by preventing the pumping of fine soils up into the coarse base aggregate while still allowing for pore water pressure dissipation from the subgrade.

A trial installation was made on a >8,000-ft low-volume county road. The purpose of the trial was to determine necessary geotextile material properties and to assess the relative performance of different pavement cross sections with and without geotextiles.

In order to assess the ability of the geotextile to survive construction, numerous samples were exhumed from beneath the two pavement types after the stone base had been completely spread and compacted, but before the surface course was constructed. Testing indicated that, under comparable conditions, like-weight woven and nonwoven geotextiles exhibit virtually the same degree of construction survivability in terms of percent strength retained. Additionally, the grade on which the installation was made has a significant influence on geotextile survivability.

The long-term performance of the installation was determined through periodic inspections of the road surface. The road surface condition was characterized and ratings were entered into the county's pavement management system (PMS) for various segments of the road. The PMS then dictates the timing of the maintenance of the various road segments. This procedure allows for the assessment of the ability to extend maintenance schedules when geotextiles are used with low-cost pavement structures. The cost savings associated with extending maintenance schedules can then be compared to the nominal additional cost of including a geotextle.

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INSTALLATION LAYOUT

Stockton Road in southern Greenville County, South Carolina, was selected for this trial installation because it had been surfaced with aggregate twice in the preceding 18 months and was once again in need of additional surfacing. This was a clear indication that the road subgrade was unstable when saturated and could benefit from the installation of a stabilization geotextile.

The full length of the road, approximately 8,100 ft, was surfaced with pavement sections as shown in Figure 1 and presented in Table 1. The following cross sections were used on approximately one-third of the road each:

- 1-in. triple treatment surface course over 3-in. compacted-stone base,
- 1½-in. asphaltic concrete (AC) surface course over 3-in. compacted-stone base, and
- 2½-in. full-depth AC binder course.

Approximately 500 ft each of three different geotextiles, 4- and 6-ounce-per-square-yard (oz/yd^2) needle-punched nonwoven geotextiles and a 4-oz/yd^2 slit-film woven geotextile, were installed between the subgrade and each pavement section. The remaining footage of the road will act as a control for the long-term evaluation of each pavement section.

Before the placement of the geotextile or pavement systems, the road subgrade was fine graded and surface saturated by water truck, and baseline cone penetration measurements were made.

Properties of stabilization geotextiles are given in Table 2.

SITE DATA COLLECTION AND EVALUATION

In order to facilitate meaningful evaluation of long-term road performance, the following information was obtained during the trial installation:

- Road centerline survey including staking of stations (50-ft intervals);

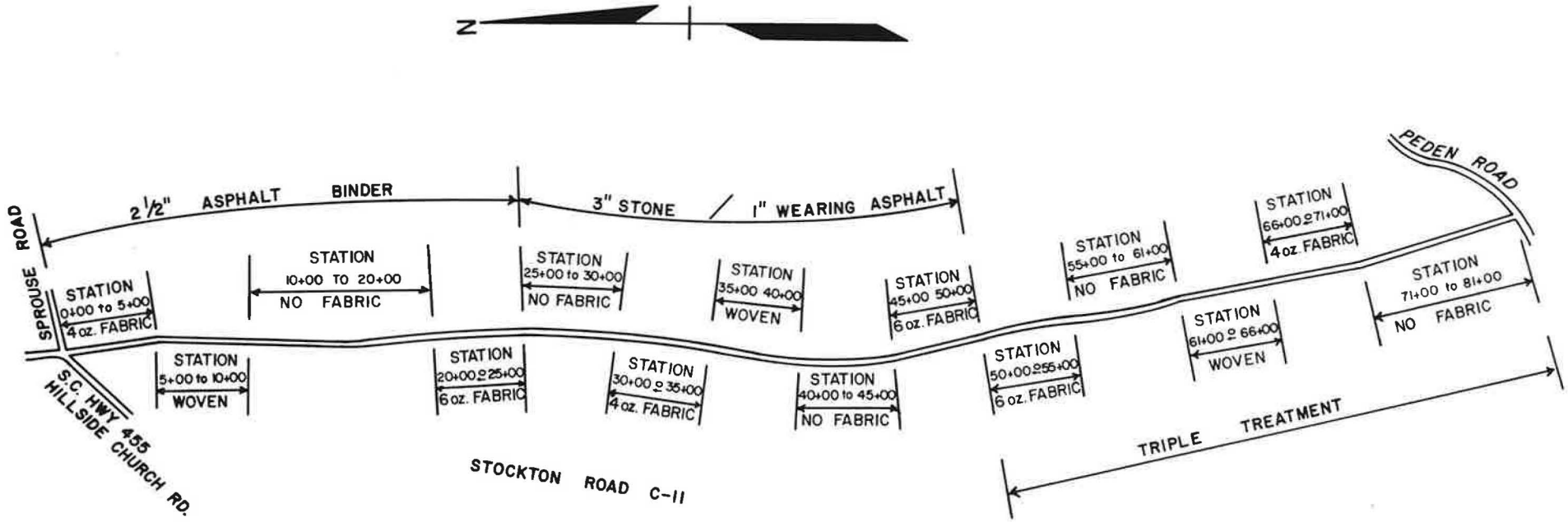

FIGURE 1 Stockton Road experimental paving plan of May 11, 1987.

TABLE 1 STOCKTON ROAD PAVEMENT INSTALLATION

Station to	Station	Stabilization Geotextile	Pavement Section
0+00	5+00	A	2 1/2" Full Depth Asphalt
5+00	10+00	B	2 1/2" Full Depth Asphalt
10+00	20+00	None	2 1/2" Full Depth Asphalt
20+00	25+00	C	2 1/2" Full Depth Asphalt
25+00	30+00	None	1 1/2" Asphalt over 3" Stone Base
30+00	35+25	A	1 1/2" Asphalt over 3" Stone Base
35+25	40+25	B	1 1/2" Asphalt over 3" Stone Base
40+00	45+50	None	1 1/2" Asphalt over 3" Stone Base
45+50	50+00	C	1 1/2" Asphalt over 3" Stone Base
50+00	55+00	C	Triple Treatment over 3" Stone Base
55+00	61+00	None	Triple Treatment over 3" Stone Base
61+00	66+00	B	Triple Treatment over 3" Stone Base
66+00	71+00	A	Triple Treatment over 3" Stone Base
71+00	81+00	None	Triple Treatment over 3" Stone Base

TABLE 2 TYPICAL PROPERTIES OF STABILIZATION GEOTEXTILES

Construction	ASTM Method	A PET Continuous Filament Needlepunched Nonwoven	B PP Silt Film Woven	C PET Continuous Filament Needlepunched Nonwoven
Weight, oz/sy	D3776	4.2	4.0	6.0
Grab Strength, lbs	D4632	135/110	200/200	205/175
Grab Elongation, %	D4632	70/85	20/18	75/85
Puncture, lbs	D3787	60	80	90
Trapezoid Tear, lbs	D4533	60/50	65/65	80/75
Mullen Burst, psi	D3786	210	385	315
Water Flow Rage, gpm/sf	D4491	140	5	130
A.O.S., sieve size	D4751	70-100	40	70-100

- Centerline plan and profile of roadway, including stations, fabric location and identification, and pavement location and identification; and
- Saturated soil strength as measured using the cone penetrometer index.

The survey revealed two segments of the roadway that would provide conditions significantly different from those of the rest of the road. These two segments involved edge drains with no outlets and steep grades. It was decided to use a heavier nonwoven geotextile in these areas and concentrate the use of the 4-oz/yd^2 geotextiles in the areas of more uniform conditions to facilitate more accurate performance comparisons of the like-weight materials.

Cone penetrometer data indicated that the road subgrade either varied in its stability over its length or, more likely, greater water penetration was achieved in portions of the roadway. Penetrometer readings were consistently in the 150 to 180 range (c = 12 to 15 psi), but from Station 0+00 to Station 57+50 they were achieved at an average penetration depth of less than 1 in. From Station 58+00 to Station 81+00, the average penetration depth was approximately 1¾ in. In either case, the upper zone of the subgrade appeared to be affected by saturation.

GEOTEXTILE INSTALLATION AND ROAD BASE CONSTRUCTION

With the road and edge drains fine graded for proper cross slope and drainage, installation of the geotextile began. The first segment to be constructed involved the construction of the stone base in two 1½-in. compacted lifts followed by a triple-treatment surface course. The thin lifts of base material with a substantial coarse fraction (see Table 3 for base aggregate grain size analysis) were expected to produce a worst-case condition on the geotextiles.

The second segment to be constructed involved the construction of a full 3-in. compacted lift of stone base followed by a 1½-in. AC surface course.

The third constructed segment was a compacted 2½-in. of full-depth AC binder course.

In all three segments, the geotextiles were unrolled and tacked in place using 16-penny nails to prevent them from being disturbed by the wind. Special caution was taken to eliminate any wrinkles in the geotextile in the full-depth segment. Because the geotextile was tacked down, only a nominal 6- to 12-in. overlap was required on the dry, firm subgrade.

Trucks dumping base aggregate were allowed to run and dump directly on the fabric. This procedure was considered a practical acceptance of the typical methods of constructing these low-volume roads as well as providing a worst-case evaluation.

A motor grader spread the aggregate to the desired depths, and an 8-ton steel-wheeled roller provided the compaction of the base material. Geotextile sampling for construction damage was done after the completion of the aggregate base, but before the construction of the surface course.

The third pavement segment was full-depth asphalt binder to a compacted thickness of 2½ in. No sampling was done in this segment, but some field observations were made.

TABLE 3 ROAD BASE AGGREGATE SIEVE ANALYSIS

50+00 to 81+00		25+00 to 50+00	
Sieve #	Percent Passing	Sieve #	Percent Passing
1 1/2	100	1 1/2	100
1	92.5	1	96.9
3/4	79.9	3/4	90.0
1/2	69.7	1/2	76.8
3/8	63.1	3/8	69.2
4	52.7	4	56.7
8	45.7	8	47.8
16	37.9	16	39.0
30	30.0	30	30.5
50	19.5	50	19.6
100	10.7	100	10.7
200	5.7	200	5.5
PAN	--	PAN	--

The only unsatisfactory observations made during the construction of the full-depth segment involved placement of the 2½ in. of hot asphalt on the woven-slit film geotextile. Circular arc–shaped cracks appeared in the pavement as the paver progressed up a modest (<1 percent) grade and once again when paving a somewhat steeper grade. This effect is believed to be a result of slippage of the pavement at the geotextile-pavement interface.

EXHUMING GEOTEXTILE SAMPLES

Geotextile sampling for determination of construction damage was performed following compaction of the aggregate base. The most severe construction loadings were assumed to occur during set-up of the base aggregate and construction of the ensuing surface course was assumed to impose less significant stresses on the geotextiles.

Geotextile sampling was not done in the full-depth asphalt pavement segment. Construction stresses in this segment only involved asphalt trucks running directly on the geotextile, which was similarly done by aggregate trucks in the other two segments, and by paver wheel loads that appeared insignificant.

Samples were initially exhumed by shoveling aggregate off 30- by 30-in. areas every 50 ft and cutting out 18- by 18-in. geotextile samples; 30- by 30-in. patches were then tucked in to repair the sampled area.

After the initial sampling of the first segment, shoveling was restricted to a doughnut around the 18- by 18-in. sample. The sample was then cut and the aggregate was gently rolled off the geotextile. It was hoped that this would minimize abrasion to the fabric caused by the sampling. Subsequently, the first segment was resampled using the doughnut approach. Laboratory results did not indicate that shoveling aggregate off the sample, rather than the doughnut approach, resulted in more abrasion to the geotextile.

All samples were marked with the station number corresponding to the sampling location and a note was made if aggregate depth above the fabric varied significantly from the desired 3 in.

LABORATORY TESTING AND RESULTS

The evaluation of construction damage of the geotextile required laboratory testing of appropriate strength properties to determine the extent of degradation resulting solely from construction-related activities.

It was decided to utilize those strength properties that are often cited in stabilization geotextile specifications but that are independent of fabric orientation. This choice simplified the notation requirements on the exhumed fabrics.

Two nondirectional tests, Mullen burst and puncture, were chosen. These tests are quick and straightforward, and each provides a useful measure of strength loss during construction.

Control samples were cut from the rolls received on site and tested in the laboratory to verify that published data were acceptable for subsequent comparisons.

Ninety-nine field samples were exhumed—22 of each geotextile in Segment 1 and 11 of each geotextile in Segment 2. Most field samples had puncture damage to a minor extent. In order to avoid extreme results, both Mullen burst and puncture tests were set up to intentionally exclude obvious puncture holes. Five Mullen burst and five puncture tests were run on each sample and the results were averaged.

As could be expected, the results were widely scattered, but when averaged for each sample and for each location, and when two samples were exhumed from the same location, the results appeared consistent.

As presented in Table 4, the 4-oz/yd^2 fabrics used in the low-survivability conditions performed similarly in terms of percent strength retained. The slit-film woven geotextile appeared to be somewhat less susceptible to reduction in puncture strength though just slightly more sensitive to Mullen burst strength reduction than the nonwoven needle-punched geotextile. The differences seem relatively insignificant and do not appear to be grounds for differentiating between the two geotextiles for purposes of construction survivability. Four-ounce-per-square-yard geotextiles are more susceptible to puncture than to abrasion under these base aggregate lifts. Although a relatively small percentage of strength reduction was apparent, puncture holes were apparent in nearly every sample.

TABLE 4 GEOTEXTILE STRENGTH RETAINED AT LOW-SURVIVABILITY CONDITIONS*

	4 oz/sy Continuous Filament Needlepunched		4 oz/sy Slit Film	
	NONWOVEN		WOVEN	
	Mullen Burst %	Puncture %	Mullen Burst %	Puncture %
Triple treatment over 3" Base	80	80	77	100
1 1/2" Asphalt Surface over 3" Base	100+	100+	93	100+

*Heavy construction equipment operating on firm, dry, well draining subgrade. Road grades are flat to slight.

TABLE 5 GEOTEXTILE STRENGTH RETAINED AT MODERATE-SURVIVABILITY CONDITIONS*

		6 oz/sy Continuouse Filament Needlepunched Nonwoven	
		Mullen Burst %	Puncture %
Triple treatment	over 3" Base	57	73
1 1/2" Asphalt Surface	over 3" Base	77	79

*Heavy construction equipment operating on poorly drained subgrade. Road grades are moderate to steep.

Table 5 presents interesting insight into the need for a more durable geotextile when more demanding survivability conditions are experienced. The 6-oz/yd^2 needlepunched, nonwoven geotextile experienced approximately 20 and 40 percent strength loss in the two pavement segments built using aggregate base. These data indicate the importance of considering road grade and drainage when assessing survivability conditions and that this geotextile may not have been durable enough for the given moderate survivability conditions.

Under both low- and moderate-survivability conditions (see Tables 4 and 5), the allowance of extraordinarily thin compacted lifts of base course, as was allowed in the triple-treatment segment of Stockton Road, should elevate the applicable geotextile survivability conditions one level (i.e., from low to moderate).

CONSTRUCTION SURVIVABILITY OBSERVATIONS AND RECOMMENDATIONS

Satisfactory performance of the roadway depends on the ability of the geotextile to survive construction without a significant reduction in the physical properties that are necessary to provide long-term separation and filtration. With retained strength as a guide, the following observations concern the construction survivability of geotextiles in low-cost, low-volume pavement structures:

- Like-weight woven-slit film and nonwoven needlepunched geotextiles exhibit the same degree of construction survivability, in terms of retained strength, under like conditions.
- The required level of survivability must include an assessment of lift thickness of base aggregate and roadway grade, as well as saturated subgrade strength and construction vehicle loading.
- 4-oz/yd^2 geotextiles of all types are too light weight to resist localized puncturing when thin base course lifts are used.

Table 6 presents survivability conditions and suggested appropriate geotextile mass per unit area.

MONITORING PAVEMENT PERFORMANCE

In order to characterize the relative long-term performance of the various pavement sections, it was necessary to periodically inspect the road surface to track degradation. Two independent visual inspection programs were initiated. The first

TABLE 6 GEOTEXTILE SPECIFICATIONS FOR CONSTRUCTION SURVIVABILITY IN LOW-COST, LOW-VOLUME ROADS*

Survivability Level	Subgrade Conditions	Base Course Thickness **	Geotextile Mass/Unit Area
Low	Dry, firm, flat	> 6"compacted	4 oz/sy
Moderate	Water sensitive, flat	> 3-4" compacted	6 oz/sy
High	Water sensitive, grade > 2%	> 3-4" compacted	8 oz/sy

* These recommendations incorporate the allowance for construction vehicles to run directly on the fabric during aggregate base construction. These recommendations expect minor puncture damage to the geotextile. The resulting greater sensitivity to pumping is not considered critical in low volume installations. Required survivability should be increased for higher volume roads to protect against puncture damage to the geotextile.

** For base course lifts less than 3", required survivability should be increased one level (i.e. low to moderate).

program involved qualitative evaluation of the surface on a quarterly and then a semiannual basis. The second program included quantitative assessments of the pavement surface on two different occasions.

The quantitative assessments were entered into a computerized pavement management system that could then project the long-term performance of each pavement segment. The qualitative surface observations were used to validate the quantitative assessments and are presented in Table 7.

PAVEMENT CONDITION EVALUATION

Greenville County used the American Public Works Association's Micro Paver Software for Pavement Management. This program was developed by the U.S. Army Corps of Engineers, Civil Engineering Research Laboratories. The basic data entered on the various pavement sections rely on surface distresses. Their quantity and severity establish the overall quality of a pavement. The pavement condition index (PCI) is established on a ranking scale from 0 to 100. The various qualitative descriptions and the relationship to the PCI numbers are presented in Table 8. In addition, the various types of distresses identified in the pavement evaluation for a PCI determination are noted at the bottom of Table 8. Each of the 19 distresses associated with asphalt pavements relates to a deduction value from the top-rank value of 100. Pavement condition information is entered into and weighting, deduction, and projecting calculations are expeditiously handled through the computer software program. A pavement is assumed to be allowed to deteriorate to a PCI of 40 before resurfacing or other rehabilitative work would be required. Pavements exposed to traffic loads and volumes significantly greater than those experienced by the low-volume roads being addressed should be maintained at some greater level. The action level is established by local preference.

TECHNICIAN TRAINING

Obviously, the key to a reliable pavement management program is the determination of the PCI value. This value relates directly to the interpretation of the road surface by the personnel conducting the evaluation. The engineering technicians conducting these evaluations went through several days of extensive classroom training followed by field visits to all types of pavement in order to get thoroughly acquainted with the various types of distresses and their severity levels. A continual cross check on work performance is made to ensure the overall quality and dependability of the program. All teams are trained such that on any given pavement section, the final PCI of that section does not vary over a range of more than five points. The results of PCI evaluations on various segments on Stockton Road are presented in Table 9.

The PCI rankings of the pavement excluded distresses that are not related to the overall structural performance of the pavement. Sample units without areas damaged by construction, utility work, and other localized distresses were intentionally selected. Sample units within each test section contained approximately 2,600 ft^2. The sample units selected were typical of the pavement within each section.

PROJECTING RATE OF DETERIORATION

Micro Paver uses a fourth-degree equation to simulate the PCI deterioration curve. Attempts were made to generate unique pavement performance curve data characteristic of Greenville County roads using PCI data that are available on all 1,390 mi of roadway within the county's inventory. The 3-in. stone base under a triple-treatment surface has long been the locally accepted standard. Samples were exhumed from numerous pavements to correlate PCI values and residual base thickness. Surface-treated pavements having PCI values of less than 20 demonstrated the near-total loss of stone-base materials. Stone-base material thicknesses ranged from 0 to 2 in. maximum, with the norm being approximately 1 in. It had been hoped that these older pavements could be used to establish a historical record of pavement performance and to generate a specific PCI deterioration curve. But a satisfactory correlation has not yet been found, and therefore a general form of the equation is being used to develop a family of curves to project pavement life (see Figure 2). An appropriate curve was then selected from the family of curves that best fit the limited performance data (PCI) through the first 2½ years. When test sections allowed, multiple sampling units were evaluated and the average PCI value was used to determine performance. Subsequent sampling of the roadway may alter the form of the general deterioration curves.

TRAFFIC VOLUMES

Before the installation of the pavements, Stockton Road served five homes. The estimated average daily traffic (ADT) was less than 50 vehicles per day (vpd). Lacking traffic count data, the general allowance of 10 vpd, per household, was used. The current actual traffic counts indicate usage at station 0 + 00 to be 300 vpd with 5 percent truck traffic. At the terminus of the project (Station 81 + 00), the traffic count is 300 vpd with 5 percent truck traffic. The road currently serves 17 residences. The paving of this roadway has had a drastic impact on the development of the area. This rural area is well removed from any area showing development trends.

PAVEMENT PERFORMANCE

As discussed earlier, geotextile/pavement sections were laid out to facilitate accurate comparisons. The 4-oz/yd^2 slit-film fabrics under full-depth asphalt developed cracking patterns 3 months after installation. These cracks have grown consistently worse with time. At the transition Station 25 + 00, the 6-oz/yd^2 fabric clearly demonstrated its ability to transmit subsurface water. At the downgradient edge, no underdrainage was provided. The alligator cracking noted at this location was attributed to subgrade deterioration caused by failure to install an adequate underdrain to remove water carried through the fabric. The same conditions prevailed at Station 55 + 00 under the triple-treatment surface.

FULL-DEPTH ASPHALT

Figure 3 shows the projected lives of the various pavement sections using 2½ in. of full-depth asphalt. In terms of pave-

TABLE 7 PAVEMENT PERFORMANCE AND OBSERVED SURFACE CONDITION

	2 1/2" Full Depth Asphalt					1 1/2" Asphalt over 3" Stone Base				Triple Treatment Over 3" Stone Base				
Date of Visual Inspection	4 oz/sy NW 0+00 to 5+00	4 oz/sy SF 5+00 to 10+00	Control 10+00 to 20+00	6 oz/sy NW 20+00 to 25+00	Control 25+00 to 30+00	4 oz/sy NW 30+00 to 35+25	4 oz/sy SF 35+25 to 40+25	Control 40+25 to 45+50	6 oz/sy NW 45+50 to 50+00	6 oz/sy NW 50+00 to 55+00	Control 55+00 to 61+00	4 oz/sy SF 61+00 to 66+00	4 oz/sy NW 66+00 to 71+00	Control 71+00 to 81+00
July 1, 1987	Complete Construction	Complete Construction	Complete Construction	Complete Construction	Complete Construction	Complete Construction	Complete Construction	Complete Construction	Complete Construction	Complete Construction	Complete Construction	Complete Construction	Complete Construction	Complete Construction
Oct. 7, 1987	Smooth, Crack-free	Substantial Slippage Cracking	Smooth Crack-free Some Raveling	Smooth, Crack-free	Smooth, Crack-free	Smooth, Crack-free	Smooth, Crack-free	Smooth, Crack-free	Smooth, Crack-free	Coarse Crack-free Slight Fines Migration	Coarse Crack-free Slight Fines Migration	Coarse Crack-free Slight Fines Migration	Coarse Crack-free Slight Fines Migration	Coarse Crack-free Slight Fines Migration
Jan. 8, 1988	Same	More Cracking	Same	Significant Hair line Cracking (Poor Drainge)	Same	Same	Same	Intermittent Transverse Cracks	Same	Same	Alligator Cracking on Steep, Grade (Wet Subgrade)	Same	Same	Same
April 18, 1988	Same	More Cracking	Same	More Hairline Cracking	Same	Same	Same	More Transverse Cracking	Same	Alligator Cracking Propagating from Control Section	Further Alligator Cracking	Same	Same	Same

TABLE 7 (*continued on next page*)

TABLE 7 (*continued*)

July 25, 1988	Same	Cracks have connected inverse Alligator Patterns	Slight Transand Longtudinal Cracking	More Hairline Cracking	Slight Transverse and Longtudinal Cracking	Same	Slight Longitudinal Cracking	More Transverse Cracking	Same	More Alligator Cracking	Severe Alligator Cracking	Same	Same	Slight Alligator and Longitudinal Cracking
Feb. 15, 1989	Same	Extensive Alligator Cracking Repressing Into Large Block Cracking	Repaired Ravelling Intermittent Transverse and minor Long. Cracking	Hairlines are now Significant Pattern of Transverse and Long Cracks	More Transverse and Longtudinal Cracking	Few Minor Longitudinal Cracks	Slight Longitudinal Cracking	Intermittent Block Cracking	Same	More Alligator Cracking	Alligator Cracking Removed and patched	Same	Same	More Alligator and Longitudinal Cracking
July 21, 1989	Minor Transverse Crack	Most Severe Alligator Patched, Blocks becoming Alligator	Intermittent Cracking Beginning to Block	Alligator Cracking Developing from transverse & Longitudinal	Large Block Cracking Develoing	Minor Cracks Appear to have healed in hot Weather	Most Cracks Appear to have healed in hot Weather	Increasing Intermittent Block Cracking	Minor Longitudinal Crack	Alligator Cracking Stabilized	Continued Alligator around Asphalt Patch	Same	Same	Severe Alligator Cracking Patched, Additional Alligator Cracks

TABLE 8 PAVING CONDITION AND PCI RANK

Pavement Condition	PCI Rank
Excellent	100
Very Good	85
Good	70
Fair	55
Poor	40
Very Poor	25
Failed	10
	0

ASPHALT PAVEMENTS
DISTRESS TYPES

1.	Alligator Cracking	*10.	Long & Trans Cracking
2.	Bleeding	11.	Patching & Util Cut Patching
3.	Block Cracking	12.	Polished Aggregate
*4.	Bumps and Sags	*13.	Potholes
5.	Corrugation	14.	Railroad Crossing
6.	Depression	15.	Rutting
*7.	Edge Cracking	16.	Shoving
*8.	Jt. Reflection Cracking	17.	Slippage Cracking
*9.	Lane/Shoulder Drop Off	18.	Swell
		19.	Weathering & Raveling

* All Distresses are Measured in Square Feet Except Distresses 4, 7, 8, 9, and 10 which are Measured in Linear Feet; Distress 13 is Measured in Number of Potholes.

ment life for the 2½-in. thickness of asphaltic binder, the performance of the 4-oz/yd² nonwoven fabric appeared to have a significant impact on pavement life. The slit-film material itself, as well as the problems associated with placing asphalt on its relatively slick surface, clearly proved to be a detriment to a full-depth asphalt pavement. The pavement section incorporating the 6-oz/yd² fabric performed unsatisfactorily. This result suggests that heavier-weight fabrics may be too spongy beneath full-depth asphalt or that lack of drainage, which was evident in this section, leads to poor performance.

ASPHALT OVER BASE COURSE

The predicted PCI values for the 1.5-in. asphaltic wearing course over the 3-in. stone base contained far less deviation than the other types of pavements and are shown in Figure 4. The 4-oz/yd² woven and nonwoven fabrics performed equally as well in this installation, whereas the 6-oz/yd² fabric performed slightly better.

TRIPLE TREATMENT OVER BASE COURSE

Figure 5 shows the projected lives of pavement sections using triple treatment over base course. Where triple treatment was provided over the 3-in. stone base, pavement life was only estimated at 7.6 years. This life is the lowest anticipated life of any of the designs used. Again, the 6-oz/yd² fabric installed under adverse conditions performed at a level less than anticipated; however, its performance under adverse conditions still exceeded the life of the pavement in the control sections. The 4-oz/yd² nonwoven fabric appeared to outperform the slit-film material. The triple treatment produces a relatively rough, coarse surface texture that is somewhat difficult to evaluate.

COMPARING PAVEMENT TYPES

Despite the effort to provide a pavement that would have a projected life of 15 years, all sections performed below expectations by showing a rather rapid decrease in quality during the first 2 years. Figure 6 shows the relative performance of the three basic pavement designs under the control conditions. The best overall performance was achieved by using a 3-in. stone base, with a 1½-in.-thick asphalt surfacing overlay. The full-depth asphalt binder material, which was expected to have equal performance characteristics, showed <9 years of projected pavement life. The porosity of the binder material may have had an impact on the rapid initial deterioration. Had

TABLE 9 PAVEMENT PERFORMANCE DETERMINED BY PAVEMENT MANAGEMENT SYSTEM USING APSA-MICROPAVER PCI AND PROJECTED PAVEMENT LIFE

	2 1/2" Full Depth Asphalt				1 1/2" Asphalt over 3" Stone Base					Triple Treatment Over 3" Stone Base				
Date of Inspection	4 oz/sy NW 0+00 to 5+00	4 oz/sy SF 5+00 to 10+00	Control 10+00 to 20+00	6 oz/sy NW 20+00 to 25+00	Control 25+00 to 30+00	4 oz/sy NW 30+00 to 35+25	4 oz/sy SF 35+25 to 40+25	Control 40+25 to 45+50	6 oz/sy NW 45+50 to 50+00	6 oz/sy NW 50+00 to 55+00	Control 55+00 to 61+00	4 oz/sy SF 61+00 to 66+00	4 oz/sy NW 66+00 to 71+00	Control 71+00 to 81+00
PCI July 1989	80	74	81									78	85	
PCI February 1990	85		76	69	84	86¦80	85¦87	85	87	70	66			67¦72
Projected Pavement Life (Yr)	10.2	7.8	8.9	7.3	11.4	11.9	11.9	11.4	12.5	7.8	7.8	8.3	8.9	7.8
Distress * Types	Light Alligator Edge Cracking	Low to Medium Block Cracking, Light Edge Cracking	Light Block Cracking, Light L&T Edge Cracking	Light Alligator Edge, L & T Cracking	Light Block, Edge & L & T Cracking	Light to Medium Edge Cracking, Light Alligator and L&T Cracking	Light Block Cracking, Edge Cracking, L & T Cracking	Light Block Cracking, Edge Cracking, L & T Cracking	Light Edge Cracking, L & T Cracking	Light Edge Cracking, Light Rutting, Raveling	Light Bumps, Edge Cracking, Rutting, Raveling	Light Bumps, Edge Cracking	Light Rutting, Raveling	Light Bumps, Light Rutting, Raveling

* L & T indicates longitudinal and transverse cracking.

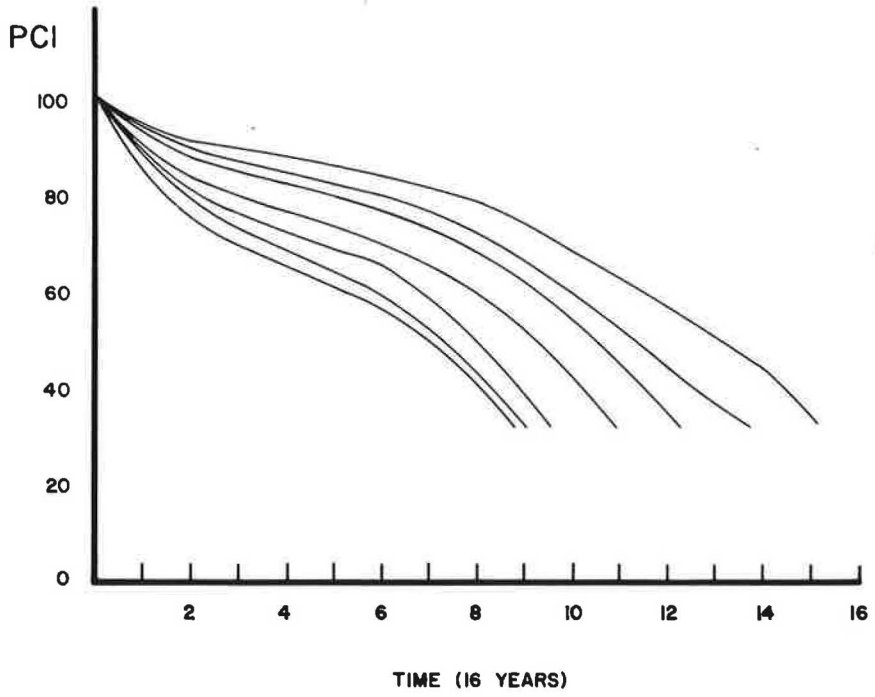

FIGURE 2 Predicted PCI family of curves.

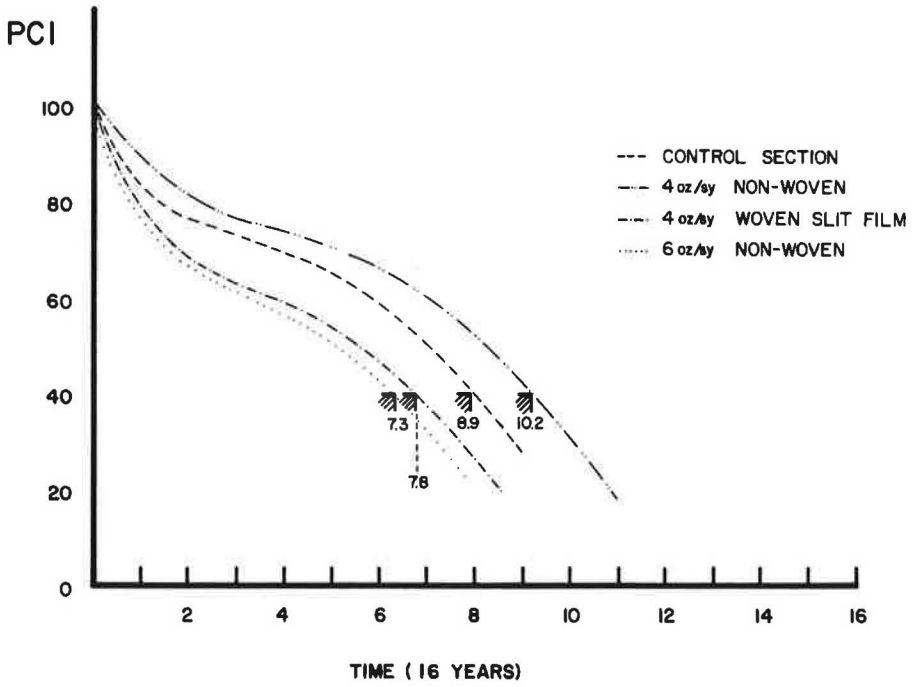

FIGURE 3 Predicted PCI curve for 2½-in. asphalt binder pavement.

the asphalt binder material performed better as an initial paving surface, it would have been a more desirable structural base for future overlays.

Although triple treatment provides an all-weather surface and protects the subgrade from moisture-related failures, overall it performs poorly as a structural material. Being the most flexible of the three designs, the rutting currently observed will likely continue and develop significant problems.

Figure 7 shows predicted PCI curves for pavements using 4-oz/yd^2 nonwoven fabrics.

COST ASSESSMENT

During the construction of the pavement section, various aspects were performed by Road and Bridge Department personnel.

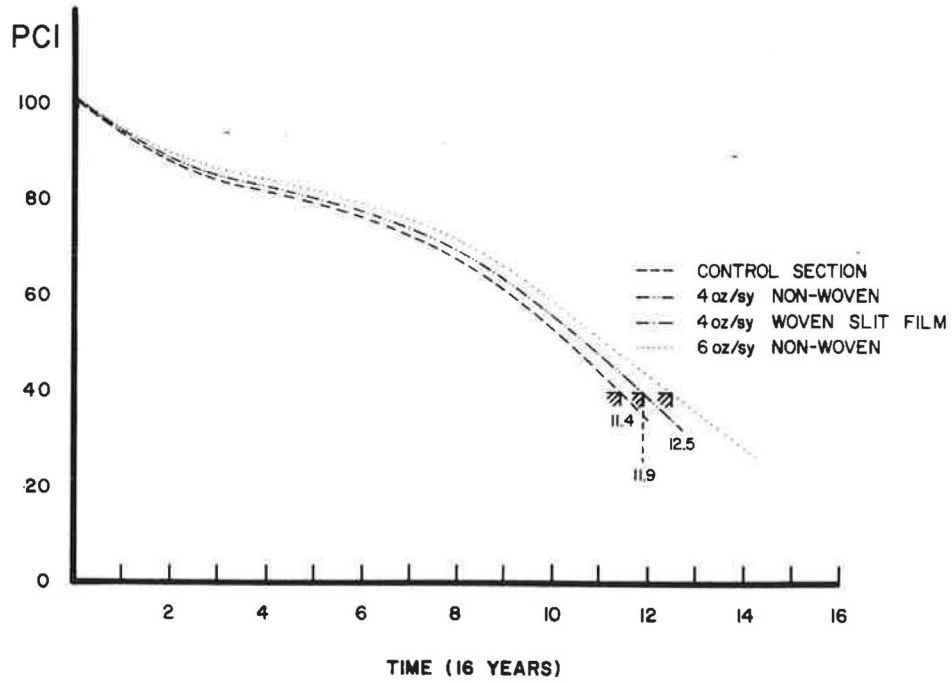

FIGURE 4 Predicted PCI curve for 1½-in. asphalt wearing course over 3-in. stone base.

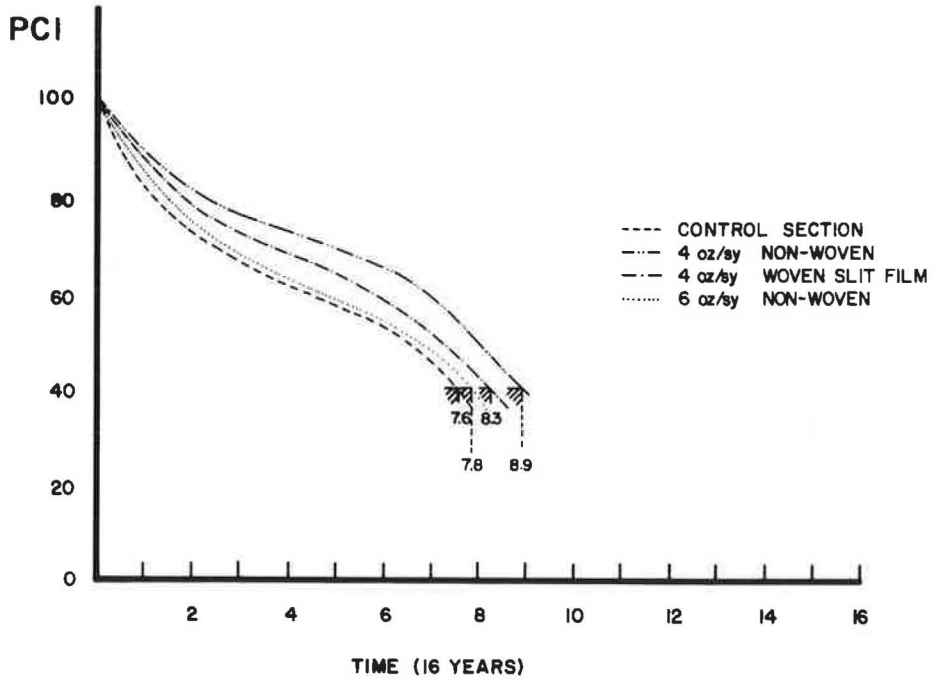

FIGURE 5 Predicted PCI curve for triple treatment over 3-in. stone base.

Costs associated with all labor, equipment, and materials were tracked using the county's work management system. The use of county-owned equipment was charged against the project on the basis of monthly rental rates for equipment. The costs of contracted services have been 10 to 15 percent lower than the unit cost for various work items performed by county personnel with equipment charges included. Table 10 presents the costs of constructing the pavements used in this test. Comparable contracted prices for similar installations are currently being experienced. All hot-laid asphalt materials, as well as the fabric and base under these materials, were placed in conjunction with the county's annual resurfacing program by a private contractor. Table 10 indicates that the actual thicknesses of applied surfacing material was 1.3 in., slightly less than the expected 1.5 in. The full-depth binder materials averaged 0.1 in. thicker than specified.

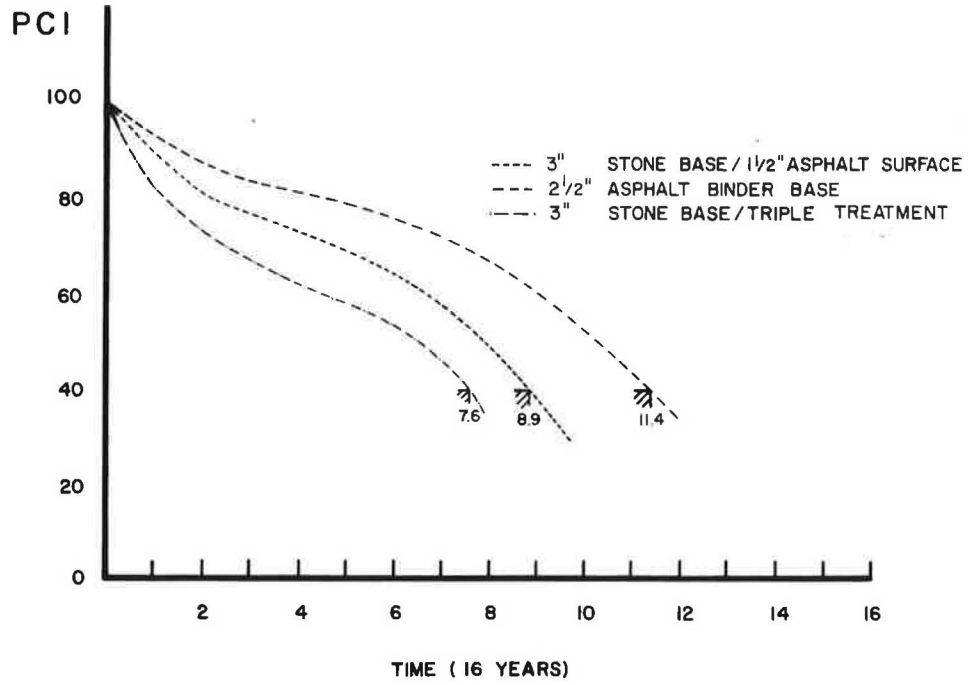

FIGURE 6 Predicted PCI curve for control pavement sections.

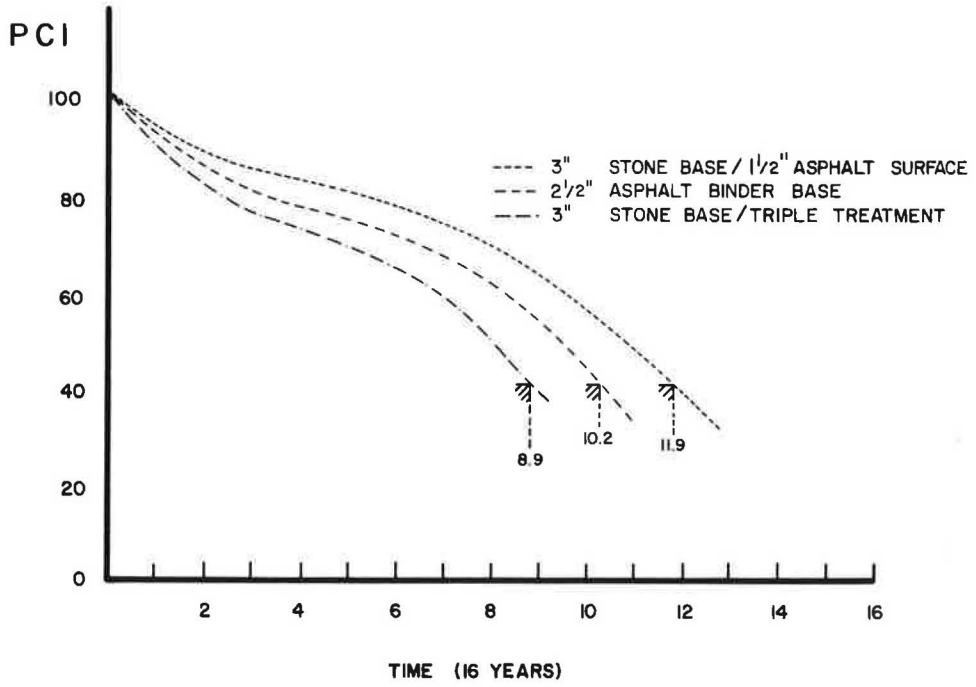

FIGURE 7 Predicted PCI curve for pavements using 4-oz/yd² nonwoven fabrics.

EQUIVALENT UNIFORM ANNUAL COSTS

In order to account for varying pavement lives when comparing pavement alternatives, an annual cost comparison technique was chosen. The equivalent uniform annual costs (EUACs) were calculated for each pavement test section using pavement lives projected by the general deterioration curves. The assumptions include an interest rate of 7.5 percent and a restoration cost of $1.78/yd² to provide an asphaltic overlay once the PCI had deteriorated to 40. EUAC equations are described elsewhere (*1*, p. 104).

Table 11 presents the EUAC values for the various pavements installed for this project. The 2½-in. full-depth binder control pavements demonstrated annual costs slightly less than

TABLE 10 STOCKTON ROAD, GREENVILLE COUNTY, SOUTH CAROLINA, ROAD SURFACING COST DATA

I. 2-1/2 FULL DEPTH BINDER (by contractor)

	FABRIC	FABRIC MATERIAL	ASPHALT(1.3" actual)	TOTAL SQUARE YARD
	$	$	$	$
w/ 4 oz/sy NW	.06	.40	3.50	3.96
w/ 6 oz/sy NW	.06	.55	3.50	4.11
w/ 4 oz/sy WSF	.06	.40	3.50	3.96
None	-	-	3.50	3.50

II. 1-1/2 ASPHALT SURFACE OVER 3" BASE (by contractor)

	FABRIC	FABRIC MATERIAL	BASE	ASPHALT(1.3" actual)	TOTAL SQUARE YARD
	$	$	$	$	$
w/ 4 oz/sy NW	.06	.40	2.29	1.78	4.53
w/ 6 oz/sy NW	.06	.55	2.29	1.78	4.68
w/ 4 oz/sy WSF	.06	.40	2.29	1.78	4.53
None	-	-	2.29	1.78	4.07

III. TRIPLE TREATMENT OVER 3" BASE

	FABRIC		BASE			TRIPLE TREATMENT			TOTAL
	LABOR	MATERIAL	LABOR	MATERIAL	EQUIPMENT	MATERIAL	LABOR	EQUIPMENT	SQUARE YARD
	$	$	$	$	$	$	$	$	
w/ 4 oz/sy NW	.06	.40	.23	1.94	.49	------1.80------		.85	5.77
w/ 6 oz/sy NW	.06	.55	.23	1.94	.49	------1.80------		.85	5.92
w/ 4 oz/sy WSF	.06	.40	.23	1.94	.49	------1.80------		.85	5.77
None	-	-	.23	1.94	.49	------1.80------		.85	5.31

TABLE 11 STOCKTON ROAD, GREENVILLE COUNTY, SOUTH CAROLINA, ROAD SURFACING COST DATA—EUAC

I. 2-1/2 FULL DEPTH BINDER (by contractor)

	INITIAL COSTS $/SY	PAVING LIFE (YEARS)	EQUIVALENT UNIFORM ANNUAL COST $/SY
4 oz/sy Non-woven	3.96	10.2	0.419
6 oz/sy Non-woven	4.11	7.3	0.500
Woven/Silt Film	3.96	7.8	0.473
Control/No Fabric	3.50	8.9	0.410

II. 1-1/2 ASPHALT SURFACE OVER 3" BASE (by contractor)

	INITIAL COSTS $/SY	PAVING LIFE (YEARS)	EQUIVALENT UNIFORM ANNUAL COST $/SY
4 oz/sy Non-woven	4.53	11.9	0.438
6 oz/sy Non-woven	4.68	12.5	0.442
Woven/Silt Film	4.53	11.9	0.438
Control/No Fabric	4.07	11.4	0.409

III. TRIPLE TREATMENT OVER 3" BASE

	INITIAL COSTS $/SY	PAVING LIFE (YEARS)	EQUIVALENT UNIFORM ANNUAL COST $/SY
4 oz/sy Non-woven	5.77	8.9	0.581
6 oz/sy Non-woven	5.92	7.8	0.620
Woven/Silt Film	5.77	8.3	0.595
Control/No Fabric	5.31	7.6	0.580

the pavement section where 4-oz/yd^2 nonwoven fabric was used. The 4-oz/yd^2 woven slit-film materials were not suitable for use under full-depth pavement.

For the pavement section constructed of 1½-in. asphaltic wearing surface over a 3-in. stone base, the annual costs for installations using fabric run slightly higher. This seems to indicate that, provided the stability of the subgrade is maintained, the presence of fabric at the interface may not be critical to overall pavement performance. After long-term performance can be monitored, it will be seen if the presence of a fabric increases pavement life by providing protection of the subgrade through its drainage characteristics.

The performance of triple treatment over the 3-in. stone base demonstrated a near-equal cost benefit for using 4-oz/yd^2 fabrics.

CONCLUSION

Geotextiles provide general stability and drainage properties that may increase the quality of a pavement section. Little is known about low-volume pavement design using geotextiles. Principles of reduction in aggregate base depths to offset costs of paving fabrics are not applicable to thin designs.

In most cases, life cycle costs for pavement with fabric were somewhat greater than the costs associated with the control sections that did not use fabrics. Local conditions still warrant the evaluation of life cycle costs associated with any project because the construction costs will vary with the locality.

Low-volume pavement designs are susceptible to accelerated deterioration. When average daily traffic is less than 500 vpd, the pavement life is significantly impacted by any increases in truck traffic. The presence of fabrics may reduce the susceptibility to rapid deterioration.

Fabric use in low-volume pavements should be regarded as a tool to maximize performance of paving materials. It is expected that the use of fabric guarantees performance of base materials at their maximum structural coefficient levels. Yet, the long-term benefits may not be seen until pavements approach 5 to 7 years of age. At these ages, the mixing of the subgrade and base materials are normally expected to begin. The retention of the subgrade and base material interface, together with the ability of the fabric to channel water out of the road section may warrant fabric installation. Fabrics may or may not enhance initial pavement performance, but as subsequent overlays are placed, fabrics continue to protect base courses from fouling and therefore will likely enhance future pavement performance.

In conclusion, the short-term results of using fabric in low-volume pavement designs is inconclusive. Future evaluations are expected to show that careful selection of appropriate fabric weights to ensure construction survivability is critical to pavement performance. Also, when engineering fabrics are used, appropriate interceptor drains must be provided at downgradient terminal fabric edges to channel water out of the pavement structure. Many questions remain regarding long-term performance of these test sections. Most notably, will fabrics prevent accelerated deterioration as cracking patterns allow water to pass through the pavement and base material to the detriment of the subgrade?

REFERENCE

1. D. G. Newman. *Engineering Economic Analysis*. Engineering Press, 1977.

Variable Tire Pressure

Effects of Variable Tire Pressure on Road Surfacings

ROBERT W. GRAU AND LEONARD B. DELLA-MORETTA

The effects of variable tire pressure on road surfacings were determined. A specially designed test road was constructed and subjected both to loaded (80,000 lb) and unloaded 18-wheeled log trucks operating in two distinct traffic lanes. The traffic was applied at tire pressures of 100 psi in one lane and at approximately 39 psi in the other lane. The background, design, construction procedure, and performance of the various sections during traffic are discussed. The results indicate that (a) when failures occurred in both lanes of the same asphalt concrete section, the ratio of 39-psi tire pressure traffic to 100-psi tire pressure traffic ranged between 1.5 and 21; (b) considerable maintenance will be required on aggregate-surfaced grades receiving high-tire-pressure unloaded traffic because of severe washboarding; and (c) the installation of central tire inflation systems that allow a driver to adjust a vehicle's tire pressure while in motion will be cost-effective for heavy trucks traveling on low-volume, low-speed roads. Savings can be realized in the reduced thickness required to withstand the loads and in the effort required to maintain aggregate-surfaced roads with trucks operating at low (39-psi) tire pressure.

The U.S. Department of Agriculture Forest Service is responsible for approximately 340,000 mi of roads in the national forests. Seventy-five percent of these roads are low-volume, one-lane, gravel-surfaced roads. In 1983, the Forest Service's San Dimas Technology and Development Center, California, began to study the effects of variable tire pressure on the cost of transporting forest products and the extent that central tire inflation (CTI) systems that allow a driver to adjust a vehicle's tire pressure while in motion will be cost-effective. Structured tests were conducted at the Nevada Automotive Test Center to quantify the effect of lower tire pressure on tire and truck performance and at the U.S. Army Engineer Waterways Experiment Station (WES) to quantify the effect of lower tire pressure on road surface deterioration and pavement thickness requirements. It is hoped that the study results will show tremendous benefits of CTI and thus encourage the development and use of CTI equipment in the logging industry. This, in turn, will result in increased income from timber sales for the Forest Service regions by reducing the effort required to maintain the roads during timber hauling. Considerable savings should be realized in the construction and maintenance of roads, by the reduction of truck and tire wear, and from the extension of haul seasons.

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SCOPE

A specially designed test road was constructed and traffic tested. The test road was subjected both to loaded and unloaded 18-wheeled log trucks operating in two distinct traffic lanes. The background, design, construction procedure, test procedures, and performance of the various test sections during traffic are described.

DESIGN

The test road was designed to determine the effect of tire pressure (deflection) on road surface deterioration and thickness design of low-volume roads. This road was approximately 0.7 mi in circumference with parallel 12-ft-wide traffic lanes. It was divided into 15 sections including curves and grades. The surfaces of these sections were constructed of native soil, crushed aggregate, or asphalt concrete (AC). The total pavement thickness or layer thicknesses were varied for each of the 15 sections in order that failures or different amounts of distress would occur throughout the proposed traffic period. A plan view of this road is shown in Figure 1. The type and thickness of the various pavements or surfacing layers for each section are also indicated in Figure 1.

PAVEMENT LAYERS

The natural soil at the test site was used as subgrade material for all sections and for surfacing of Section 3. The soil has a liquid limit of 32 and a plasticity index of 12 and classifies as a lean clay (CL) according to the united soil classification system (*1*). Classification data are shown by Curve 1 in Figure 2.

A 4- to 8-in.-thick crushed aggregate base course was specified for six of the nine AC-surfaced test sections. Five sections were surfaced with 3- to 9-in. layers of the same crushed aggregate. The material used (crushed limestone) met all requirements set forth in EM 7720-100R (*2*) for base or surface courses, Grading D. A grading curve for this material is shown as Curve 2 in Figure 2.

A mix design for the AC surfacing layer placed on nine of the test sections was in accordance with that required by the Louisiana Department of Transportation and Development (*3*). The combined grading curve for the AC mixture and the gradation specification limits are shown in Figure 3. The job mix limits are percent asphalt, 4.5 to 4.7; stability, 1,200-lb minimum; flow, 15 maximum; percent voids total mix, 3 to 5; and percent voids filled, 71 to 80.

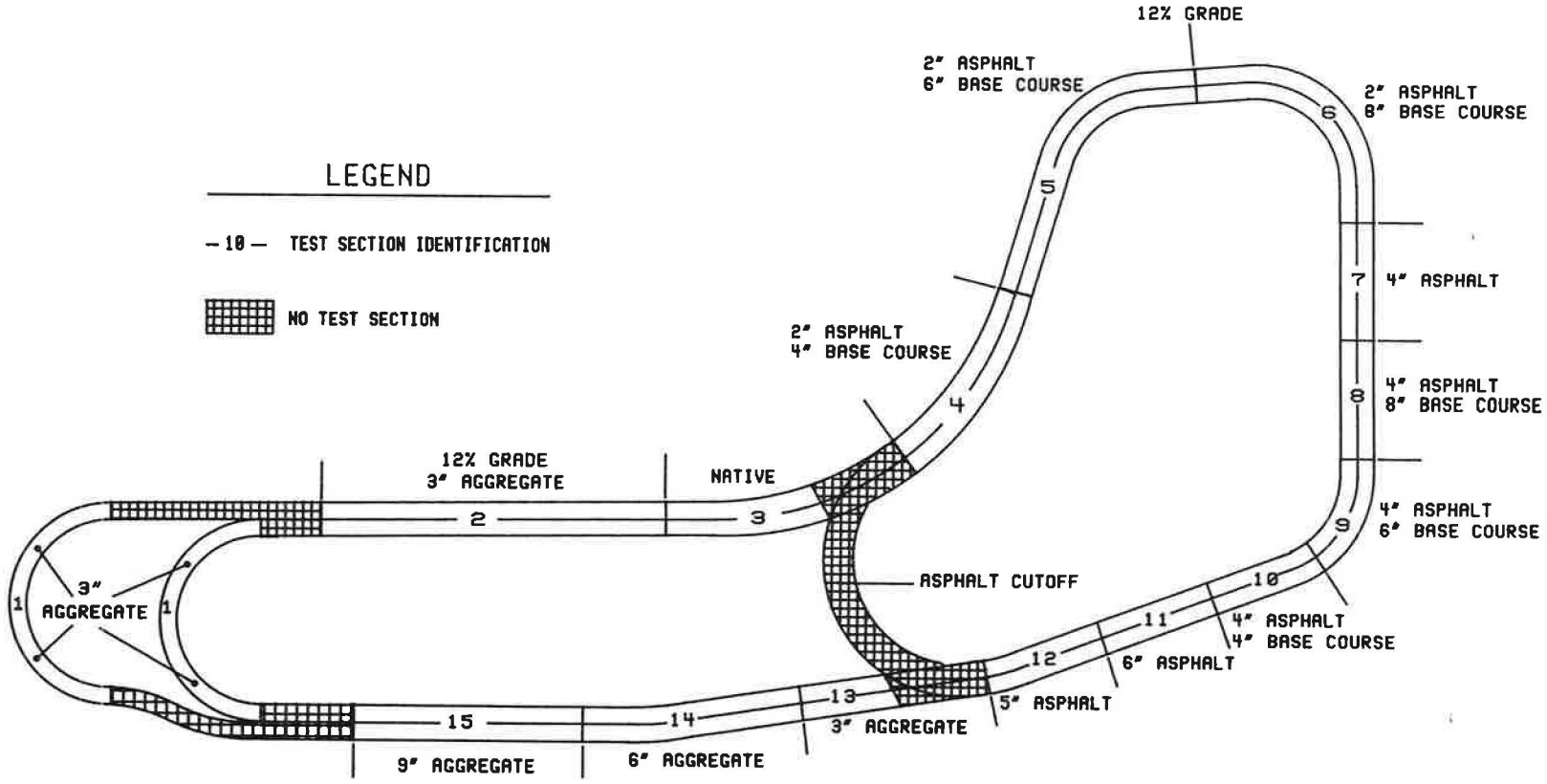
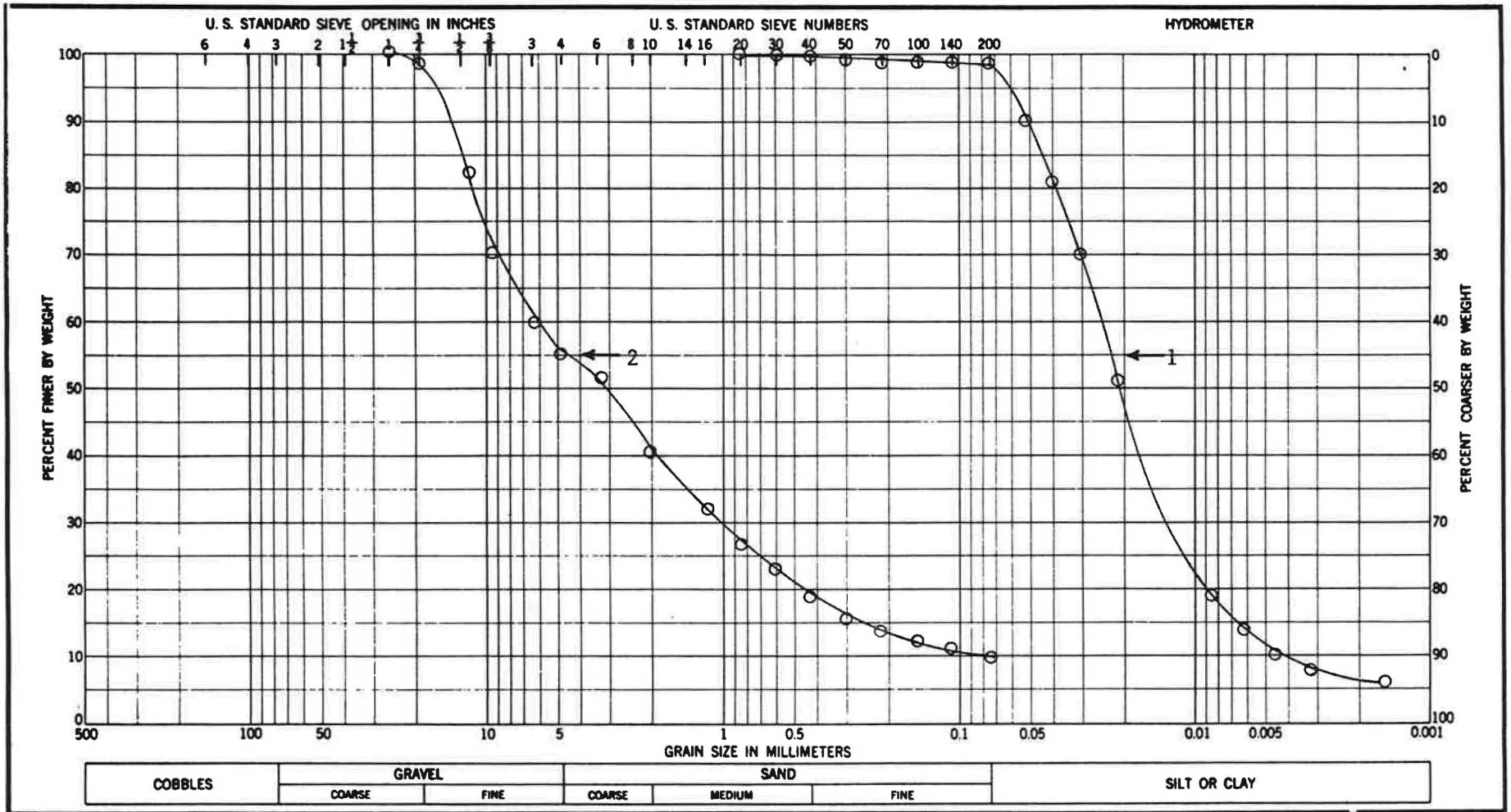


FIGURE 1 Plan view of test road.



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
1		LEAN CLAY (CL)		32	20	12
2		CRUSHED LIMESTONE (SW-SM)		15	11	4

GRADATION CURVES

Project

Area

Boring No.

Date

ENG FORM 1 MAY 63 2087

FIGURE 2 Classification data for subgrade and base materials.

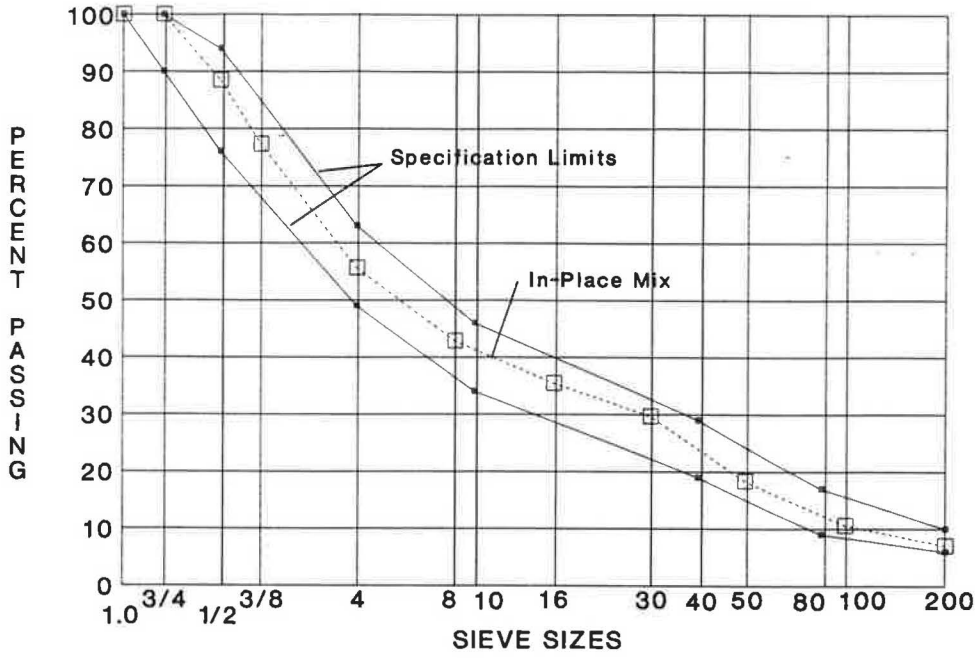


FIGURE 3 Combined gradation curve and gradation specification limits for AC.

CONSTRUCTION

The test road was constructed at WES during the period September 1986 to September 1988. All work was accomplished under contract by Lewis Miller Construction Company, Inc., Vicksburg, Mississippi.

Before subgrade construction, trees were removed and vegetation stripped from a 100- to 150-ft-wide path along the centerline of the test road. Subgrade construction consisted of rough grading and placement and compaction of fill material to at least 90 percent of AASHTO T180 (*4*) to an elevation 1 ft below finished subgrade along the entire roadway. Borrow material was then brought in and the entire roadbed was elevated 1 ft. The borrow and existing subgrade material were the lean clay described earlier. The borrow material was hauled and spread with self-loading scrapers into an 8- to 10-in. loose lift. The desired density, 95 percent of AASHTO T180, was achieved by compacting with a sheepsfoot and a self-propelled rubber-tired roller. Water content and density tests were performed on each lift of all sections to verify that the desired density had been obtained. After the final lift had been compacted and met density requirements, it was fine bladed to the desired elevation and sealed with the rubber-tired roller.

The crushed limestone material used for the base layer of the AC sections and surfacing layer of the aggregate-surfaced sections was placed with an asphalt finisher to prevent segregation. A single-drum vibratory roller was used to compact this material. Generally, the desired density, 95 percent of AASHTO T180, was obtained. After compaction, the sections to be surfaced with AC were primed with approximately 0.30 gal/yd^2 of MC–70 cutback asphalt.

The AC hot mix was placed with an asphalt finisher in two 12-ft-wide lanes and one 10-ft-wide lane. The 2-in. pavement was placed in one lift, whereas the 4- and 5-in.-thick pavements were placed in two layers. The 6-in. AC in Section 11 was placed in three 2-in.-thick layers. Compaction of each layer was obtained by breakdown rolling with a tandem steel-wheeled roller, followed by a self-propelled rubber-tired roller. Before laydown, samples were obtained from the mix for laboratory extraction, gradation, and compaction tests to ensure uniformity and compliance with the job-mix design. The properties of the mix, as presented in Table 1, were within the job-mix limits, except for the percent total voids, which averaged 2.8. Three percent was the lower limit of the voids total mix. After compaction, cores were cut and tested for density. Results of these tests (Table 1) indicate that the percent density of the in-place mix to the laboratory-compacted specimens was 97.5, which is slightly less than the 98 percent required.

A summary of the as-constructed (0 pass data) thickness, California bearing ratio (CBR), water content, and density of the various pavement layers in each test section is presented in Table 2. The field data measurements were obtained from test pits excavated from the surface of the pavement to 6 in. below the surface of the subgrade. The test pits were located in a nontraffic area along the centerline of the roadway at about the midpoint of each test section. The relatively low CBR values (21 to 47) for the crushed stone base and surfacing material were caused by the thin (3- to 7.5-in.) layers of this material being placed over the subgrade, which had CBR values ranging between 7 and 22. Pumping was observed during compaction of the crushed aggregate material, which implies deflection and yielding of the subgrade under the roller, and thus, the inability to obtain the desired density and resulting strength. During construction, only an average of 93.8 percent of T180 was obtained in the crushed aggregate.

TABLE 1 PROPERTIES OF BITUMINOUS MIXTURES

Test Sec.	Asphalt Content % Total Mix	Stability (lb)	Flow 1/100 (in.)	Percent Voids Total Mix	Percent Voids Filled	Unit Weight Total Mix (pcf)	% Plant Laboratory Density
			Plant-Mixed Laboratory Compacted Samples[a]				
--[b]	4.4	2,736	11	2.8	78.3	147.9	--
--	4.9	2,319	12	2.4	82.4	147.8	--
--	4.4	2,359	12	3.1	76.5	147.5	--
			Field Cores[c]				
4	--	--	--	--	--	145.5	99
	--	--	--	--	--	145.4	99
	--	--	--	--	--	142.2	96
5	--	--	--	--	--	143.3	97
	--	--	--	--	--	146.5	99
	--	--	--	--	--	145.5	99
6	--	--	--	--	--	141.8	96
	--	--	--	--	--	144.8	98
	--	--	--	--	--	141.7	96
7	--	--	--	--	--	141.7	96
	--	--	--	--	--	145.2	98
	--	--	--	--	--	144.4	98
8	--	--	--	--	--	144.7	98
	--	--	--	--	--	143.5	97
	--	--	--	--	--	146.5	99
9	--	--	--	--	--	145.9	99
	--	--	--	--	--	144.2	98
	--	--	--	--	--	144.5	98
10	--	--	--	--	--	145.0	98
	--	--	--	--	--	143.6	97
	--	--	--	--	--	140.0	95
11	--	--	--	--	--	142.9	97
	--	--	--	--	--	140.5	95
	--	--	--	--	--	143.1	97
12	--	--	--	--	--	146.4	99
	--	--	--	--	--	141.3	96
	--	--	--	--	--	144.4	98

[a]Specimens compacted by the 75-blow Marshall method.
[b]A dash (--) indicates not applicable.
[c]Field cores obtained before traffic testing.

TESTING AND BEHAVIOR UNDER TRAFFIC

Test Vehicles

Traffic was applied to the test road with the two 18-wheeled log trucks, shown in Figure 4, running in separate test lanes (for low and high pressures) around the test track. Test traffic included both loaded and unloaded passes of the log trucks. Table 3 presents the weights of each side of the five axles under each loaded truck and the total gross weight per truck. All wheels on the trucks were equipped with 11R24.5 XZY, Load Rating G, 14-ply tires. One truck was operated at typical highway pressure of 100 psi in all tires, which resulted in tire deflections of about 7 and 10 percent when unloaded and loaded, respectively. The low-pressure truck operated at a constant tire deflection (21 percent), which required tire pressures of approximately 25 and 39 psi for the unloaded and loaded conditions, respectively. Tire pressures were set each morning before traffic was begun.

Test Lanes

The test road was divided into two lanes (for high and low tire pressure) and two loops (the entire test road and the AC sections only). The high-pressure truck operated in the outside lane at all times, and the low-pressure truck operated in the inside lane. Each truck was operated in its respective wheel path at all times, which resulted in four distinct wheel paths across the roadway. The wheel paths are numbered one through four with Wheel Path 1 located in the high-pressure lane and adjacent to the outside edge of the roadway and

TABLE 2 SUMMARY OF MOISTURE, DENSITY, AND CBR MEASUREMENTS

Test Sec.	Lane	Material	Thickness, in. Design	Thickness, in. Actual	Passes	Depth (in.)	CBR (%)	Water Content (%)	Dry Density, pcf In-Place A	Dry Density, pcf AASHTO[a] T 180 B	Percent T 180 Density A/B
1	L	Aggregate	3	4.8	0	0	47	1.4	128.5	135.4	95
		Subgrade	--[b]	--		0	12	18.4	102.8	109.1	94
					0	6	7	20.5	100.0	104.1	96
	H	Aggregate	3	4.0	0	0	23	1.6	124.4	135.8	92
		Subgrade	--	--		0	8	18.5	105.2	108.9	97
						6	25	16.5	106.5	113.3	94
2	H&L	Aggregate	3	3.8	0	0	22	1.7	125.4	136.0	92
		Subgrade	--	--		0	15	20.1	99.5	105.1	95
						6	10	18.5	103.5	108.9	95
3	H&L	Subgrade	--	--	0	0	18	17.3	103.0	111.6	92
						6	25	18.1	106.8	109.8	97
4	H&L	AC	2	2.5	0	--	--	--	--	--	--
		Aggregate	4	3.8		0	25	3.5	131.4	142.6	92
		Subgrade	--	--		0	7	19.2	106.7	107.2	100
						6	13	21.4	100.2	101.7	99
	H	AC	2	2.0	158	--	--	--	--	--	--
		Aggregate	4	2.0		0	11	4.6	136.9	143.6	95
		Subgrade	--	--		0	6	19.6	105.5	106.3	99
						6	7	22.5	101.5	99.0	103
	H	AC	2	3.5	323	--	--	--	--	--	--
		Aggregate	4	3.5		0	23	134.0	3.6	143.0	94
		Subgrade	--	--		0	7	108.4	17.2	111.8	97
						6	15	102.9	19.0	107.7	96
	H	AC	2	3.0	323	--	--	--	--	--	--
		Aggregate	4	3.5		0	32	143.2	4.5	143.7	100
		Subgrade	--	--		0	18	104.3	19.0	107.7	97
						6	8	104.1	19.6	106.3	98
5	H&L	AC	2	2.7	0	--	--	--	--	--	--
		Aggregate	6	5.8		0	21	129.1	3.4	142.3	91
		Subgrade	--	--		0	10	105.6	18.5	108.9	97
						6	14	97.4	16.5	113.3	86
	L	AC	2	3.5	2,076	--	--	--	--	--	--
		Aggregate	6	4.5		0	33	145.5	4.1	143.6	101
		Subgrade	--	--		0	3	104.8	19.4	106.7	98
						6	8	100.5	20.6	103.8	97
	H	AC	2	2.3	1,414	--	--	--	--	--	--
		Aggregate	6	6.0		0	20	142.6	4.9	143.1	100
		Subgrade	--	--		0	3	103.2	20.7	103.6	100
						6	6	100.2	21.0	102.8	97
6	H&L	AC	2	2.3	0	--	--	--	--	--	--
		Aggregate	8	5.5		0	24	137.4	3.4	142.3	97
		Subgrade	--	--		0	16	104.6	18.1	109.8	95
						6	10	106.0	18.3	109.3	97
	H	AC	2	2.5	1,104	--	--	--	--	--	--
		Aggregate	8	6.5		0	52	149.0	4.6	143.6	104
		Subgrade	--	--		0	4	105.7	17.8	110.4	96
						6	6	102.6	19.4	106.7	96
	L	AC	2	2.5	2,076	--	--	--	--	--	--
		Aggregate	8	7.0		0	46	143.3	4.7	143.5	100
		Subgrade	--	--		0	6	103.9	19.1	107.5	97
						6	8	100.4	19.9	105.5	95
7	H&L	AC	4	5.2	0	--	--	--	--	--	--
		Subgrade	--	--		0	22	109.1	15.8	114.6	95
						6	19	106.3	17.6	110.9	96

TABLE 2 (*continued on next page*)

TABLE 2 (*continued*)

Test Sec.	Lane	Material	Thickness, in. Design	Thickness, in. Actual	Passes	Depth (in.)	CBR (%)	Water Content (%)	Dry Density, pcf In-Place A	Dry Density, pcf AASHTO T 180 B	Percent T 180 Density A/B
8	H&L	AC	4	5.0	0	--	--	--	--	--	--
		Aggregate	8	6.3		0	34	130.7	2.3	138.6	94
		Subgrade	--	--		0	16	108.1	16.2	114.0	95
						6	10	107.8	17.4	111.4	97
9	H&L	AC	4	4.7	0	--	--	--	--	--	--
		Aggregate	6	5.5		0	44	130.2	2.4	138.8	94
		Subgrade	--	--		0	19	108.0	16.3	113.8	95
						6	17	103.1	16.8	112.6	92
10	H&L	AC	4	4.3	0	--	--	--	--	--	--
		Aggregate	4	3.6		0	39	129.6	2.3	138.6	94
		Subgrade	--	--		0	20	105.9	15.4	115.4	92
						6	21	103.7	18.4	109.1	95
11	H&L	AC	6	5.7	0	--	--	--	--	--	--
		Subgrade	--	--		0	16	104.3	16.3	113.8	92
						6	15	100.2	19.0	107.7	93
12	H&L	AC	5	4.7	0	--	--	--	--	--	--
		Subgrade	--	--		0	16	100.3	17.8	110.4	91
						6	27	100.0	17.2	111.8	89
13	H&L	Aggregate	3	3.0	0	0	35	133.4	2.8	139.6	96
		Subgrade	--	--		0	15	103.9	18.9	107.9	96
						6	31	104.4	17.4	111.4	94
14	H&L	Aggregate	6	5.8	0	0	32	135.0	2.8	139.6	97
		Subgrade	--	--		0	12	103.0	20.2	104.8	98
						6	12	100.9	17.8	110.4	91
15	H&L	Aggregate	9	7.5	0	0	32	129.0	3.3	141.9	91
		Subgrade	--	--		0	17	107.8	18.1	109.8	98
						6	22	104.7	18.3	109.3	96

[a]Based on AASHTO T 180 maximum density at field-in-place water content.
[b]A dash (--) indicates not applicable.

FIGURE 4 Test vehicles.

TABLE 3 TEST TRUCK CHARACTERISTICS

Axle	Weight, lb		
	Left	Right	Total
	High Pressure Truck, 100 psi		
Steering	4,920	4,670	9,590
Front Drive	8,625	8,180	16,805
Rear Drive	8,230	8,200	16,430
Front Trailer	7,835	8,520	16,355
Rear Trailer	8,415	9,005	17,420
Gross Vehicle Weight			76,600
	Low Pressure Truck, 39 psi		
Steering	4,880	4,650	9,530
Front Drive	8,635	8,675	17,310
Rear Drive	8,450	8,565	17,015
Front Trailer	8,265	8,115	16,380
Rear Trailer	8,655	8,500	17,155
Gross Vehicle Weight			77,390

Wheel Path 4 in the low-pressure lane and near the inside edge of the test track. The loaded traffic ran in a counterclockwise direction, whereas unloaded traffic ran in the clockwise direction. This traffic pattern resulted in simulating actual forest harvest conditions in that the loaded trucks are traveling down an aggregate grade and the unloaded trucks up the grade. During the trafficking of the aggregate sections, loaded and unloaded traffic were alternated on a daily basis until significant data were obtained to determine the difference in performance. Loaded traffic was then applied full time. When measurements or repair was required on the aggregate portion of the test road, the asphalt cutoff was used and only the AC portion or Sections 4 to 12 trafficked with loaded trucks. A summary of the total traffic applied is presented in Table 4.

Failure Criteria

In judging failure of the AC test sections, the performance of the surface course and underlying layers was considered. Base course and subgrade failures caused by shear deformation were anticipated because it was not possible to apply a heavy compaction effort in the thinner pavement sections. The term "shear deformation" refers to excessive plastic movement or, in the extreme, the rupture of any element in the pavement structure. This behavior was evident when severe rutting and longitudinal cracking of the surface course were observed. Rut depths are defined as the maximum vertical distance from the bottom edge of a straightedge placed on the shoulders (upheaval) of the rut to the bottom of the rut. Shoving was also a major type of distress observed during the trafficking of the test section. Shoving occurred in the outside wheel path of a horizontal curve and could be detected by either the outward movement of the total thickness of AC or of the top layer in relation to the underlying layer.

Because hot bituminous mixes are placed to provide a smooth riding surface and to waterproof the base against the penetration of surface water, a pavement section was considered failed when any of the following conditions occurred:

1. Surface rutting of 2 in. or more along a continuous 20-ft-long rut,
2. Surface cracking to the extent that the pavement was no longer waterproof, or
3. Severe shoving resulting in 2-in.-deep ruts, or severe cracking of the AC surface.

Aggregate is placed on a gravel-surfaced road to protect the subgrade from being overloaded and to make the surface more resistant to the abrasive effects of traffic. The ability of the aggregate layer to carry heavy sustained traffic mainly depends on the thickness of the layer. Reduction in thickness such as that caused by rutting decreases the load-carrying capacity of an aggregate-surfaced road. Gravel roads require considerable maintenance such as blading and dust control to correct rutting and washboarding. Ruts are defined and measured for gravel roads, as described earlier, for AC-surfaced roads. Washboarding is a series of closely spaced ridges and valleys perpendicular to the direction of traffic and at fairly regular intervals. Washboarding is measured in inches and is the ver-

TABLE 4 SUMMARY OF TEST TRAFFIC

Pavement Type	High Pressure Lane, passes		Low Pressure Lane, passes	
	Loaded	Unloaded	Loaded	Unloaded
Asphalt	6,764	1,113	8,333	1,385
Aggregate	2,645	1,112	3,089	1,384

tical distance between the top of the ridge and bottom of the valley. The aggregate sections were considered failed when any of the following conditions existed in a 20-ft-long section of a wheel path:

1. Three-inch ruts in test Sections 1, 2, and 13;
2. Four-inch ruts in test Sections 14 and 15; or
3. Washboarding of 3 in. or more.

The 3-in. rut depth failure criterion was used for Sections 1, 2, and 13 because the total thickness of the aggregate layer was only 3 in. A greater degree of rutting in these sections would result in the bottom of the rut possibly being below the aggregate-subgrade interface. Grading would then result in a soil aggregate layer rather than an aggregate surface. After several grading cycles, the original aggregate would be of little benefit structurally or as a surfacing material because of contamination.

BEHAVIOR OF PAVEMENT UNDER TRAFFIC

Visual observations of the behavior of the test sections were recorded throughout the traffic test period of each lane. These observations were supplemented by photographs. Level readings, nondestructive testing using a falling weight deflectometer, condition surveys, roughness measurements, asphalt strain, deflections at various depths in selected AC sections, drop cone penetrometer readings, pavement temperature, and various climatic data obtained from an on-site weather station were recorded before and at intervals during traffic to show the development of pavement distress. After failure, a thorough investigation was made by excavating test trenches across the wheel paths to observe the various layers in the structure along with CBR measurements and other pertinent tests in these layers. General observations, rut measurements for all sections, and maintenance frequencies for the aggregate sections taken during traffic are discussed in the following paragraphs. Tables 5 and 6 present the maximum rut depths and general descriptions for the failures occurring in the AC and unsurfaced sections, respectively. An in-depth analysis of the measurements and data recorded during the conduct of this investigation is scheduled to be completed in December 1991.

Asphalt-Surfaced Sections

At the beginning of traffic, slight pumping or vertical movement of the pavement surface was observed as the high-pressure truck traversed the 2-in.-thick AC sections. Surface deformation and pumping were most evident in the outside wheel paths of the horizontal curves. After about 71 passes, distinct rutting and hairline longitudinal cracking along the outside edges of Wheel Path 2 were noticed in Section 4. As traffic continued, the rutting and cracking in Section 4 became more severe; one area was rated as failed after 158 passes. Figure 5 shows the general view of this initial failure in the AC portion of the rest road. The maximum rutting in this area was 2.3 in. deep, and the cracking had progressed into alligator cracking. After an additional 165 passes, for a total of 323 passes, a second failure occurred in Section 4, Wheel Path 2. Again, the majority of the rutting occurred in Wheel Path 2, and little distress had occurred in Wheel Path 1. Wheel Path 1 in Section 4 is located in the inside portion of the curve. This failure is attributed to overloading or failure of the subgrade. Figure 6, which is a view of a test pit excavated in the failed area, indicates severe rutting of the subgrade. At the time of this second failure in the high-pressure lane (323 passes), little distress was present in the low-pressure lane. No cracking was evident, and the maximum rut depth was only 0.5 in. The rate of pavement deterioration with traffic decreased after the second failure. The next failures occurred after 1,104 and 1,414 passes in Wheel Path 1 of Sections 6 and 5, respectively. When these sections were judged failed in the high-pressure lane, only hairline cracking and minor rutting were detected in the low-pressure lane. The average rut depth in the low-pressure lane of Section 6 after 1,104 passes was about 0.7 in., and in Section 5 about 0.4 in. after 1,414 passes. Throughout the remainder of traffic in the high-pressure lane (6,764 loaded passes), only two additional failures occurred—one in Section 5, Wheel Path 1, after 2,210 passes and another in Section 10, Wheel Path 1, also at 2,210 passes. The mode of failure in Section 5 was the same as that of the previous failures, and the Section 10 failure was attributed to severe shoving of the top lift of AC after 1,812 passes (see Figure 7), which later developed into severe cracking and rutting of the surface. The failure in Section 10 occurred in an area where the traffic was entering a horizontal curve

TABLE 5 SUMMARY OF RUT DEPTH MEASUREMENTS TAKEN IN AC FAILURES

Test Section	Lane	Wheel Path	Number of Passes	Maximum Rut Depth, in.	Remarks
4	H	2	158	2.3	Severe cracking
4	H	2	323	7.0	Severe cracking
6	H	1	1,104	8.3	Severe cracking
5	H	1	1,414	4.9	Severe cracking
6	L	3	2,076	4.0	Severe cracking
5	L	4	2,076	7.5	Severe cracking
5	H	1	2,210	6.0	Severe cracking
10	H	1	2,210	9.0	Severe shoving and cracking
4	L	3	3,324	3.3	Severe cracking
4	L	4	3,845	3.8	Severe cracking

TABLE 6 SUMMARY OF RUT DEPTH MEASUREMENTS TAKEN IN AGGREGATE FAILURES

Test Section	Lane	Wheel Path	Number of Passes - Loaded/ (Unloaded)	Maximum Rut Depth (in.)	Degree of Wash-boarding	Remarks
1	H	2	58	4.0	None	Failed; overlaid with 12 in. of aggregate
2	H	1	58	4.8	Low	Failed; overlaid with 12 in. of aggregate
3	H	1	58	3.5	None	Failed; overlaid with 12 in. of aggregate
1	L	3	66	5.0	None	Failed; overlaid with 12 in. of aggregate
2	L	4	66	3.0	None	Failed; overlaid with 12 in. of aggregate
3	L	3	66	4.5	None	Failed; overlaid with 12 in. of aggregate
2	H	1	60 (112)	6.9	Moderate	Failed; section graded
2	H	1	282 (259)	5.0	Severe	Failed; section graded
2	H	1	282 (282)	4.5	Severe	Failed; section graded
2	H	1	282 (342)	4.5	Moderate	Failed; section graded
2	H	1	584 (556)	4.0	Severe	Section graded
13	H	1	883 (672)	4.3	Low	86 passes in rain; rutting increased 3.1 in.; graded
13	L	4	1,077 (838)	4.8	Low	98 passes in rain; rutting increased 3.3 in.; graded
14	H	2	883 (672)	5.8	Low	86 passes in rain; rutting increased 4.0 in.; graded
14	L	3	1,077 (838)	5.0	Low	98 passes in rain; rutting increased 3.2 in.; graded
15	H	1	883 (672)	4.3	Low	86 passes in rain; rutting increased 1.5 in.; graded
15	L	4	1,077 (838)	3.5	Low	98 passes in rain; rutting increased 1.2 in.; graded
13	H	2	1,331 (1,112)	2.3	Low	Section graded prior to watering
13	L	3	1,600 (1,384)	1.4	Low	Section graded prior to watering
14	H	1	1,331 (1,112)	2.0	Low	Section graded prior to watering
14	L	3	1,600 (1,384)	1.8	Low	Section graded prior to watering
15	H	2	1,331 (1,112)	1.6	Low	Section graded prior to watering
15	L	4	1,600 (1,384)	2.5	Low	Section graded prior to watering
14	H	2	2,015 (1,112)	4.9	Severe	Failed; section graded
15	H	2	2,103 (1,112)	4.0	Severe	Failed; section graded
15	H	1	2,321 (1,112)	>4.0	Moderate	Failed
15	H	1	2,544 (1,112)	4.9	Moderate	Failed

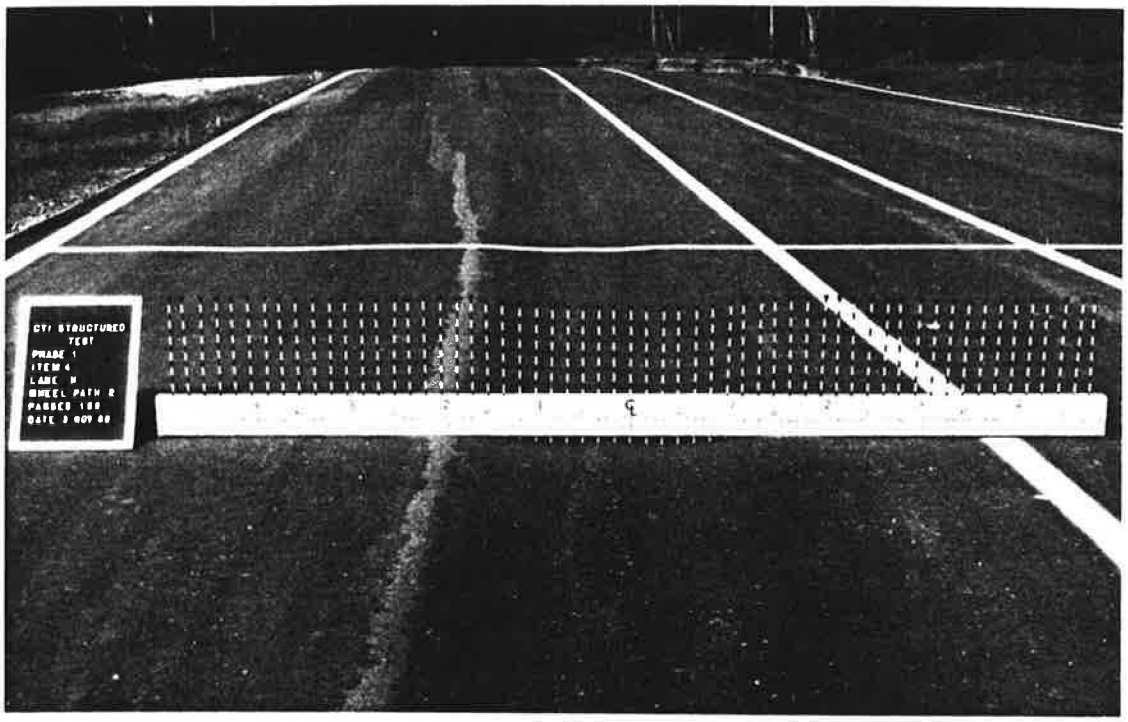


FIGURE 5 High-pressure lane, Section 4, general view of initial failure, 158 passes.

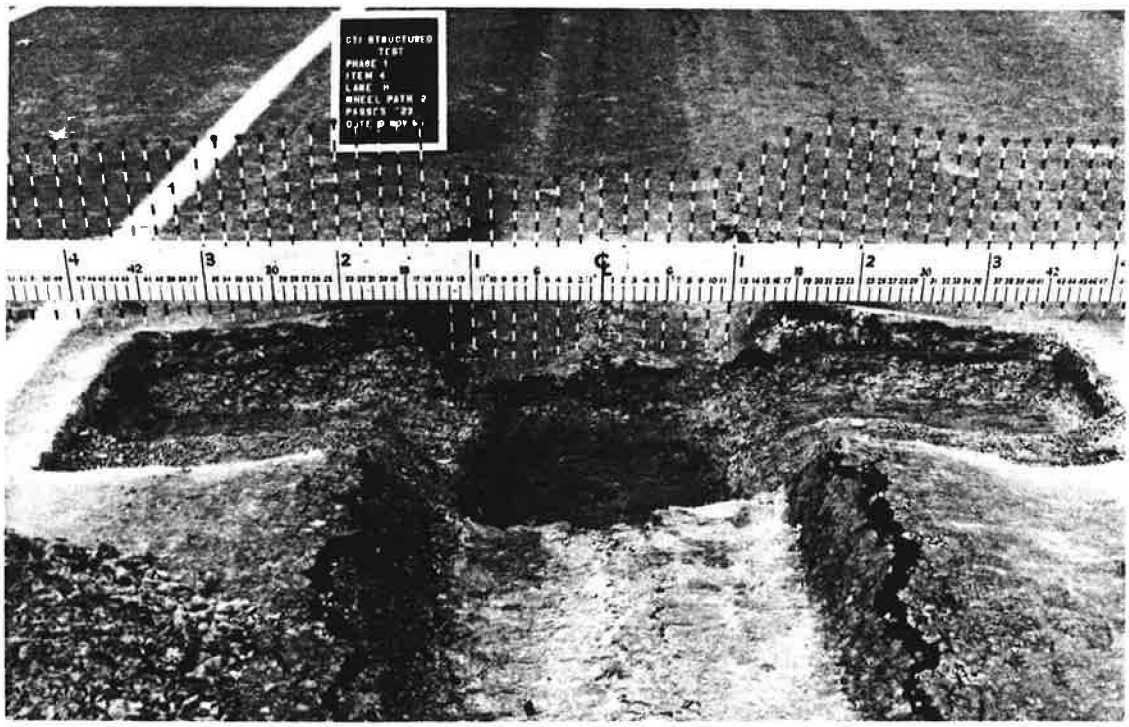


FIGURE 6 High-pressure lane, Section 4, test trench after 323 passes.

FIGURE 7 High-pressure lane, Section 10, shoving after 1,812 passes.

FIGURE 8 Low-pressure lane, Section 10, low-severity shoving after 4,200 passes.

and at the beginning of warm-weather traffic. Test traffic was applied to the AC section during two periods—October 1988 to February 1989 and June to November 1989. The shoving, shown in Figure 7, occurred after 245 passes had been applied during June 1989. Shoving (Figure 8) was detected in the low-pressure lane of Section 10 after 1,200 passes of warm-weather traffic, or a total of 4,200 passes.

Four failures, two in Section 4 and one each in Sections 5 and 6, occurred in the low-pressure lane. The failures in Sections 5 and 6 were judged failed after 2,076 passes and those in Section 4 after 3,324 and 3,845 passes. Severe cracking and rutting were the mode of each of these failures. A general view of the portion of Section 5 judged failed after 2,076 passes is shown in Figure 9.

Although the same criteria were used in judging failure in both traffic lanes, the rutting at failure in the high-pressure lane appeared more pronounced. Generally, there was little upheaval associated with the ruts in the low-pressure lane. Also, the ruts caused by the low-pressure truck were 6 to 12 in. wider than those in the opposite lane.

Aggregate-Surfaced and Native Section

Traffic was begun after several days of rain, and it was soon evident that Sections 1 to 3 would withstand few passes of the 80,000-lb log trucks. Rutting was observed in both lanes of these three test sections after one pass. After 58 passes in the high-pressure lane and 66 passes in the low-pressure lane, traffic was discontinued because of 2-in.-deep or greater ruts in the wheel paths of these three sections. Subgrade strength measurements were not made in Sections 1 to 3 at this time because a downpour occurred as traffic was stopped. Figure 10 shows a view of the low-pressure lane of Section 1 at failure. The only noticeable difference in the performance of the two lanes in Sections 13 to 15 at this time was low-severity washboarding in the high-pressure lane of Section 15 (see Figure 11), as compared to none in the other sections. It was decided that because of the weak subgrade beneath Sections 1 to 3, little additional information could be gained by blading and applying more test traffic. Therefore, these sections were overlaid with enough aggregate (a 12-in.-thick layer) to bridge the weak subgrade and withstand the scheduled test traffic. During the remainder of traffic, these sections were monitored to determine the effect of tire pressure on maintenance requirements. As traffic was continued, loaded and unloaded traffic were alternated on a daily basis. Little distress was noticed during the first day of traffic, which totaled 60 and 72 passes of loaded trucks over the high- and low-pressure lanes, respectively. The next day, the unloaded trucks operated and washboarding was noticeable in Section 2 in the high-pressure lane after about 50 passes. The corrugation was 2.5 in. deep, and the truck driver reduced speed to maneuver safely over Section 2. After 112 passes, severe washboarding was measured throughout the high-pressure lane of Section

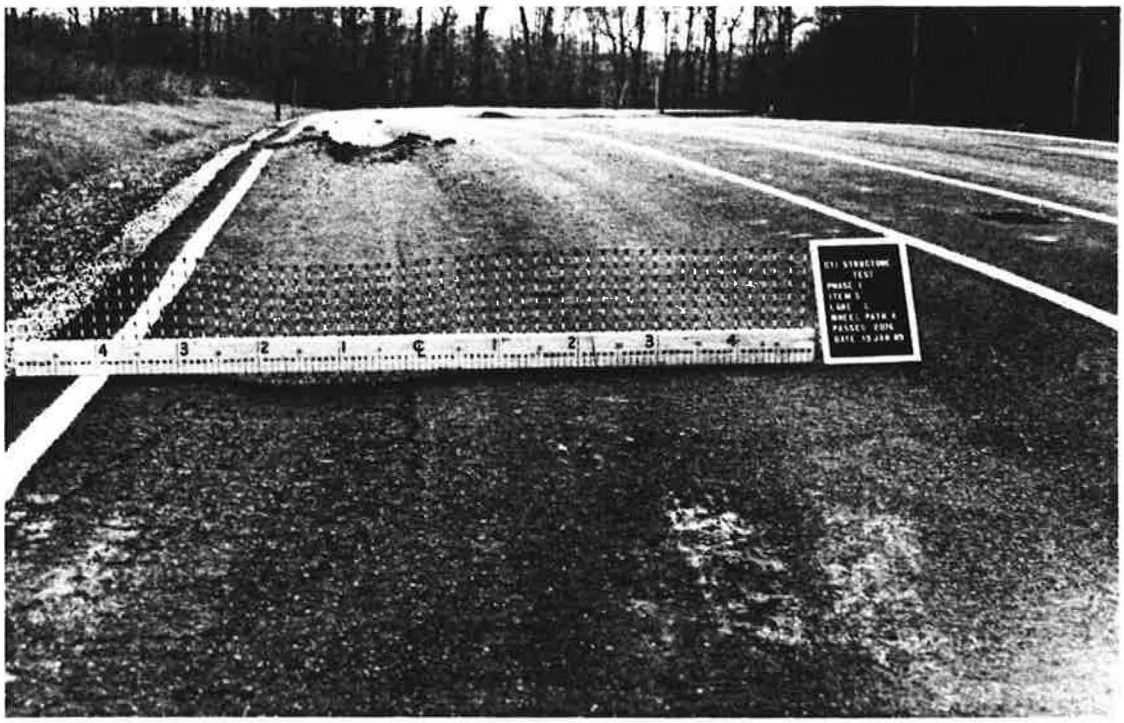


FIGURE 9 Low-pressure lane, Section 5, failure after 2,076 passes.

FIGURE 10 Low-pressure lane, Section 1, failure after 66 passes.

FIGURE 11 High-pressure lane, Section 15, low-severity washboarding, after 58 passes.

2. At this time, this section was judged as failed and required blading. Unloaded traffic was applied directly after blading, which again resulted in washboarding. After 10 passes, the high-pressure truck again had to reduce speed, and after 35 passes the corrugations were 2.5 in. deep. When traffic was switched to loaded, the maximum depth of washboarding decreased about 1 in. The high-pressure unloaded traffic continued to cause severe distress in Section 2, and after 584 loaded and 556 unloaded passes this section was graded for the fourth time. Performance at this section was then discontinued. Little distress was observed in the low-pressure lane of Section 2, and grading was never required. The greatest distress occurred when a pothole developed in the no-test-section area adjacent to this section, which resulted in washboarding migrating into Section 2. However, as shown in Figure 12, the low-pressure tires seemed to dampen out the bouncing of the truck, which resulted in no corrugation 15 to 20 ft from the pothole. The horizontal curves in Section 1 performed approximately the same under low- and high-pressure traffic. Neither lane required grading after being overlaid. The high-pressure lane received 2,586 and 1,172 passes of loaded and unloaded traffic, respectively, whereas the low-pressure lane received a total of 3,023 loaded and 1,384 unloaded passes of the log truck. By the end of traffic, the only distress observed was minor rutting in both lanes and low-severity corrugation in the high-pressure lane.

Because of the higher subgrade strengths and no vertical or horizontal curves, Sections 13 to 15 performed better during traffic than did Sections 1 to 3. Little distress was detected in Sections 13 to 15 until about 90 passes of loaded traffic was applied to both lanes during a light rain. This traffic caused considerable increase in rutting of all wheel paths of each test section. Each section was rated as failed because of severe rutting and required grading. These failures were attributed to a wet subgrade, and there was no notable difference between the performance of the various items or test lanes. After drying of the subgrade and grading, traffic was continued using loaded and unloaded trucks. Test traffic (loaded and unloaded) was only applied during dry conditions. Little distress was observed with traffic being applied under these conditions; therefore, it was decided to discontinue the unloaded traffic. After several days of loaded traffic, the rut depths in all wheel paths of Sections 13 to 15 averaged about 1.5 in. with little indication of increasing. An irrigation system was then installed to simulate rainfall and weaken the pavement structure at a controlled rate. As water was applied, drop cone penetrometer measurements were taken to monitor the aggregate and subgrade strengths. As traffic was applied, failure was reached once in the high-pressure lane of Section 14 and three times in Section 15 because of rutting and severe washboarding. No failures were recorded in the low-pressure lane. Figure 13 shows the high-severity washboarding in the high-pressure lane of Section 14 at failure. For comparison purposes, a view of the maximum distress in the low-pressure lane of Section 14 at the same time the high-pressure lane was rated failed is shown in Figure 14.

SUMMARY OF FINDINGS AND RECOMMENDATIONS

The findings from the traffic testing of the CTI test road indicated the following:

1. The failures and distresses in the high-pressure lane of the AC sections were more pronounced than those in the low-pressure lane.

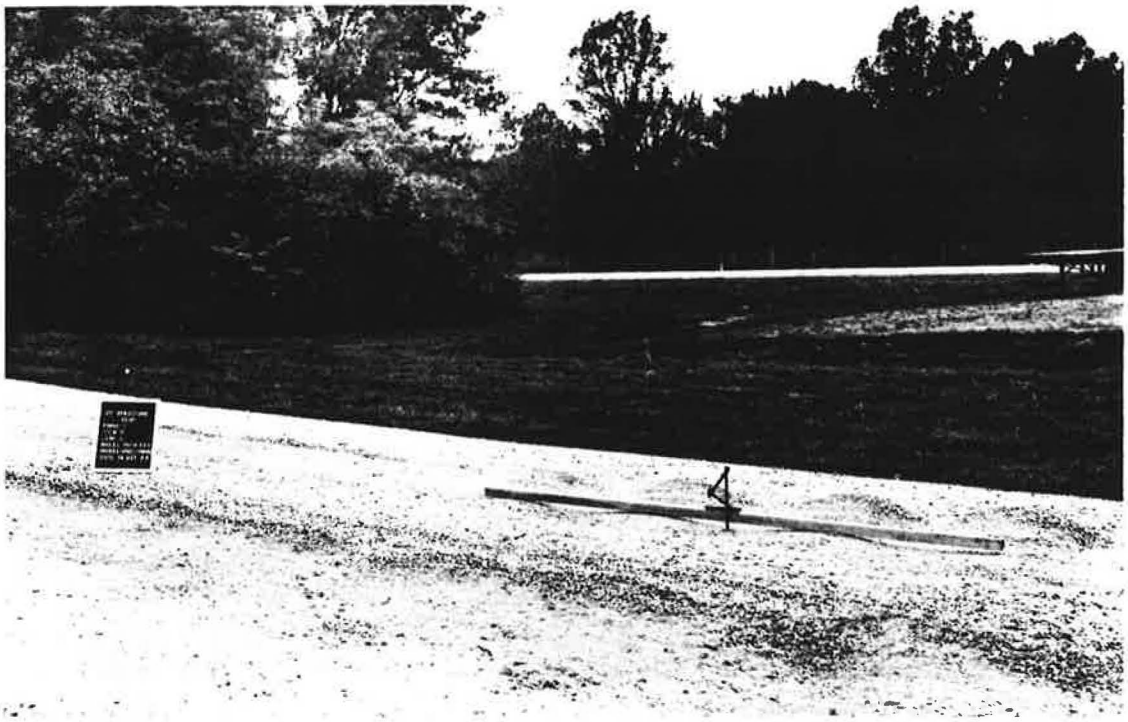


FIGURE 12 Low-pressure lane, Section 2, long intervals between corrugations.

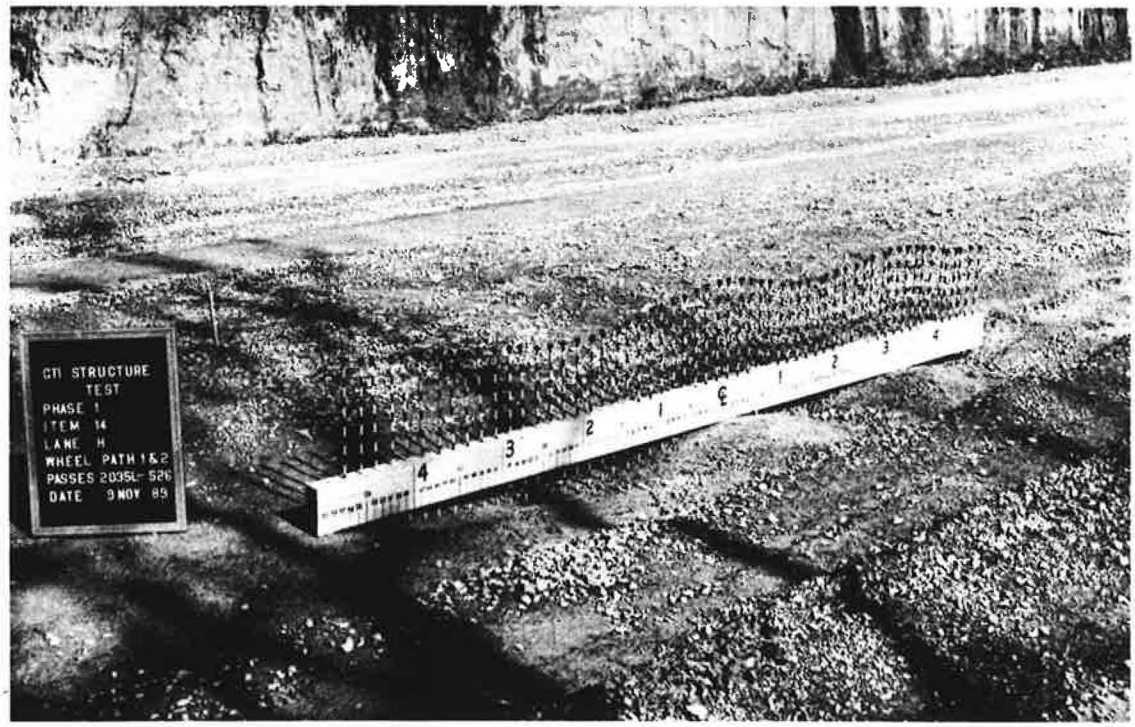


FIGURE 13 High-pressure lane, Section 14, high-severity washboarding.

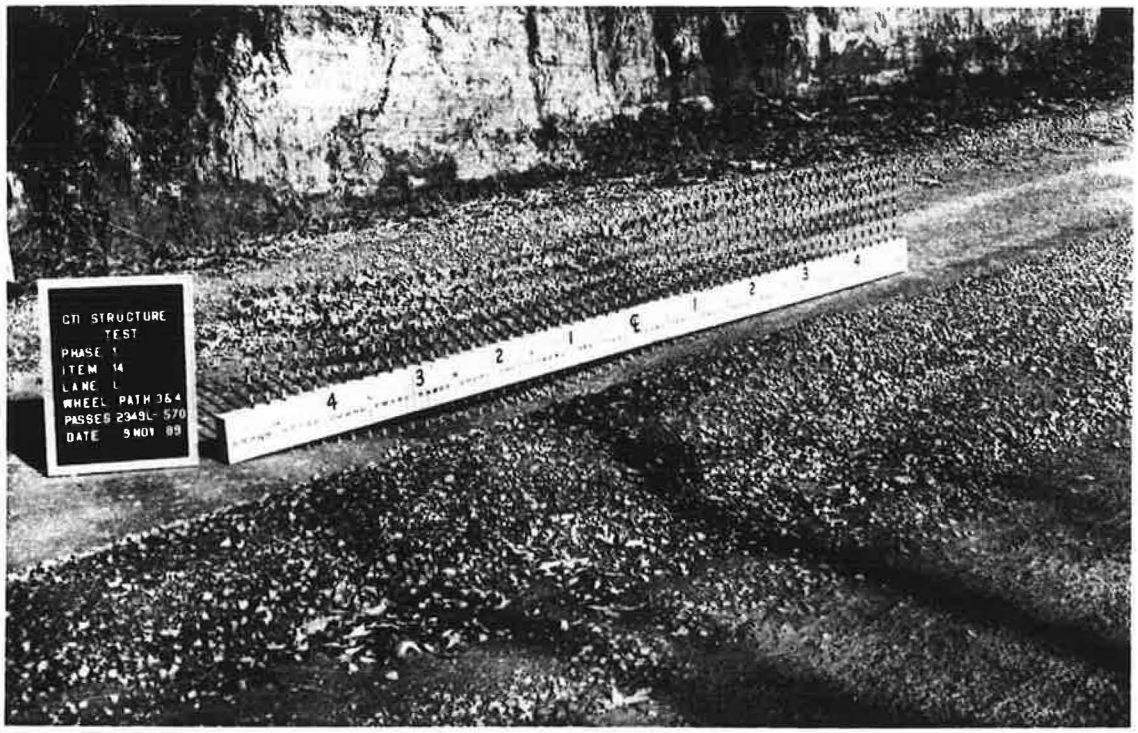


FIGURE 14 Low-pressure lane, Section 14, low-severity depression.

2. When failures occurred in both lanes of the same AC section, the ratio of low-pressure to high-pressure traffic to initial failure ranged between 1.5 and 21.
3. More raveling was observed in the low-pressure lane in the horizontal curves of the AC sections than in the high-pressure lane.
4. Comparative pavement performance of the thicker AC sections is unavailable because traffic was stopped before failures occurred.
5. The first failures, which occurred in Sections 1 to 3 and 13 to 15, should not be considered in the analysis of the test results. These failures occurred in both lanes after the same amount of traffic had been applied directly after a rain. All of these failures were attributed to subgrade failure.
6. Considerable maintenance will be required on aggregate-surfaced grades receiving high-pressure unloaded traffic because of the severe washboarding. This type of distress is not a factor under low-pressure traffic.
7. There was no appreciable difference in the performance of aggregate-surfaced horizontal curves because of different tire pressures.
8. The performance of the straight and flat aggregate sections was considerably better in the low-pressure lane compared with the high-pressure lane.

On the basis of the performance or lack of performance of the 15 sections under the loading conditions described, the following recommendations are made:

1. Additional traffic should be applied. Only one failure occurred in the thicker AC sections; and little comparative data, with the exception of some maintenance data, were obtained during the trafficking of the aggregate sections.
2. At a minimum, the existing test results should be normalized and analyzed.
3. Considerations should be given to alternating the loaded and unloaded traffic after 10 passes or whatever the normal interval is when harvesting a forest when, and if, additional traffic is applied. Continuous unloaded traffic of the high-pressure truck was severe on the aggregate-surfaced sections.

ACKNOWLEDGMENT

The authors gratefully acknowledge the support of the Forest Service, FHWA, U.S. Army Corps of Engineers, and WES. This paper is published with the permission of the Chief of Engineers.

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The views expressed herein are those of the authors, who are responsible for the facts and accuracy of the data. The contents do not necessarily reflect official views or policies of the Forest Service, FHWA, U.S. Army, or WES.

Reduced Tire Inflation Pressure—A Solution for Marginal-Quality Road Construction Rock in Southeast Alaska

BILL POWELL AND BRUCE BRUNETTE

When marginal-quality rock is encountered in Southeast Alaska, the rock often does not provide adequate support for truck haul. The road surface tends to rut and the rock continually breaks down after heavy repeated wheel loads combined with wet conditions. This process reduces the gravel to fine silt and clay-sized particles that will not support construction vehicles. The traditional solution has been to blade this material off the road and haul additional higher-quality rock to support the traffic. This procedure results in higher costs and additional stream sediment. By using radial tires with lowered tire pressures, the road surface became more compact with repeated wheel loads. This result has produced large savings, exceeding $500,000 on one project, and this concept is expected to provide future contract savings for road building and logging activities.

High-quality, durable rock is available for most road contracts in Southeast Alaska, but in some areas only poor- to marginal-quality rock borrow materials are available for road construction. This fact can result in a road surface too weak to support truck haul, especially in the wet climate and soft, highly organic, subgrade conditions encountered on the Tongass National Forest. Immediately after placement, the marginal-quality rock breaks down with repeated applications of heavy construction traffic, resulting in deep ruts within the wheel path. The conventional solution typically involves blading the decomposed aggregate slurry off the roadway, and importing better-quality materials from a distant source. This solution increases construction costs significantly and also increases the amount of potential stream sediment delivery. An alternative solution is described that was used on a timber access road construction project, the Toncan timber sale, that used reduced constant tire pressure and radial tires. Although conditions described here are representative of southeast Alaska, they may also apply to other areas where poor- to marginal-quality road surfacing material, combined with high rainfall, are encountered.

TYPICAL ROAD CONSTRUCTION

In Southeast Alaska, the typical road construction process involves preparing the subgrade with a hydraulic backhoe that levels the saturated subgrade material to the pioneer-grade elevations. Because the saturated excavated material is usually too weak to be incorporated into the road embankment, and rain frequency makes drying impractical, most of the roadway excavation is wasted. During pioneer operations, the hydraulic backhoe incorporates the clearing debris, such as limbs, tree tops, cull logs and stumps, into the prepared subgrade as a debris-reinforced mat. Once the subgrade is prepared, pit-run quarry rock is end-dumped on the debris-reinforced mat and spread with a bulldozer. The depth of rock on the finished road typically ranges between 30 and 48 in., except occasional weak muskeg may require more rock, sometimes exceeding 10 ft. Road surface failures can be caused either by inadequate subgrade support or the degradation of rock borrow materials. Photographs of the construction of the Toncan timber sale road, which is typical in Southeast Alaska, are shown on Figures 1–3.

PROJECT HISTORY

The Toncan timber sale is located approximately 20 mi southwest of Petersburg, Alaska, on Kupreanof Island, as shown in the key map of Figure 4. The project involved harvesting approximately 50 million board feet (mmbf) of timber and the construction of 21.5 mi of permanent, 16-ft-wide, low-standard, low-volume roads. During the early planning stages of the Toncan timber sale, it was recognized that for the first 8.5 mi, the rock barrow materials available for construction were of marginal quality, on the basis of past road construction in the vicinity. As a result, an intensive geotechnical field evaluation was done to identify the better-quality sites along the proposed road construction corridor. When the timber sale contract was prepared, the best of the marginal-quality borrow sources were designated and incorporated into the contract. Because how these marginal materials would perform on the road was still uncertain, construction personnel were notified that rock quality problems could be anticipated. The site map of the Toncan timber sale shown in Figure 5 indicates the planned road location and the area of the sale, where only marginal borrow materials were available for construction.

The Toncan timber sale was awarded in the spring of 1986 to Mitkof Lumber Company, of Petersburg, Alaska. Road

B. Powell, USDA Forest Service, Juneau, Alaska. B. Brunette, USDA Forest Service, Petersburg, Alaska.

FIGURE 1 Road subgrade preparation.

FIGURE 2 End-dumping rock used for road construction.

FIGURE 3 Bulldozer used to shape desired road prism.

construction for the project began in the spring of 1987, and continued throughout the drier summer months. The two initial quarries contained suitable rock; however, when the third quarry site was developed in late July, Forest Service and contractor personnel had serious reservations that the material could sustain heavy haul once the fall rainy season began. The contract called for the development of approximately 40,000 yd^3 of material from quarry Site 3.

Rock quality testing results met Forest Service standard specifications, but the material tended to break down when it became wet, and the aggregate did not support the heavy construction traffic being used to build the road. The fine-grained, highly fractured, phyllite schist historically has created problems on previous road construction projects in southeast Alaska. Both authors of this paper have experienced similar situations in the Cascade Mountains of Oregon and Washington, where rock quality problems occurred when

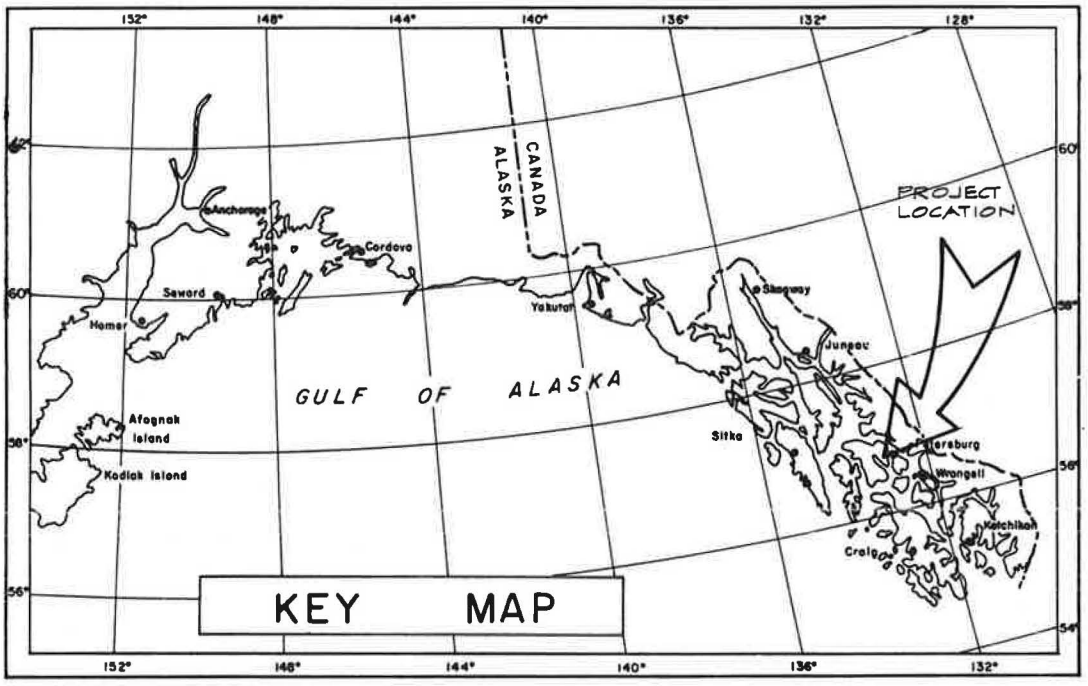

FIGURE 4 Key map.

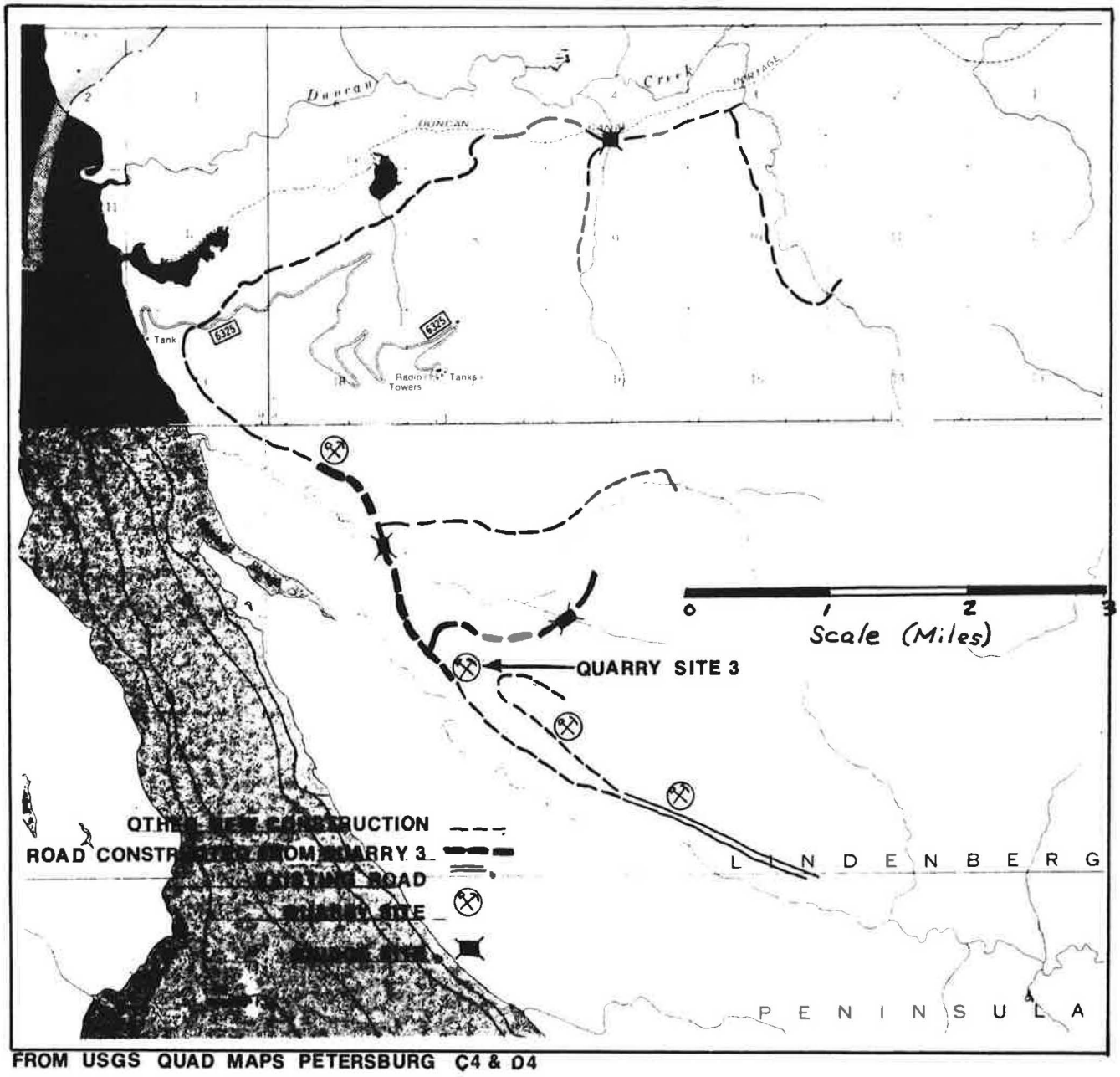

FIGURE 5 Toncan timber sale area.

material testing results narrowly passed Forest Service standard specifications. Materials testing for rock quality is presented in Table 1.

Gradation testing was performed on the pit run material immediately after placement and after several days of haul. The purpose of the testing was to specifically determine what percentage of mechanical breakdown was occurring through abrasion during rock haul. The results of gradation testing are displayed in Table 2.

Severe rock degradation was experienced during rock haul. This process, concurrent with heavy rainfall, resulted in 12- to 18-in.-deep ruts, excessive sedimentation, and daily blading was required to continue operations. During some periods of heavy rainfall, haul operations could not continue for an entire day without performing maintenance because of the poor performance of the material. Figure 6 shows the conditions of the rock after the initial placement, and Figure 7 shows the conditions during rock haul.

CONSIDERATIONS FOR PROJECT MODIFICATIONS

On the basis of studies that were conducted by the Forest Service in California, the use of radial tires, central tire inflation (CTI), and reduced tire pressure significantly reduced road maintenance and road surface damage. This fact was initially reported by Dela-Moretta (*1*) in 1984, when a loaded western-style log truck was operated on the Klamath National Forest with tubeless radial tires at a tire pressure of 25 psi.

TABLE 1 MATERIAL TEST RESULTS FOR ROCK BORROW MATERIAL FOR TONCAN TIMBER SALE

	Los Angles Abrasion (AASHTO T 96)	Durability Index (AASHTO T 210) Coarse	Fine	Sand Equivalent (AASHTO T 176)
	%	%		%
Specification	40 maximum	35 min.	35 minimum	35 minimum
Test Result	29	41	73	40

TABLE 2 MATERIAL GRADATION BEFORE AND AFTER TRUCK HAUL

	Finer than 7mm (3inch)	Finer than #4 sieve 4.7mm (sand size)	Finer than #200 sieve .074mm
	%	%	%
Before Haul	100	17	3
After Haul	100	30-50	17

In controlled testing conditions, Hodges (*2*) compared road damage by operating identical western-style log trucks, one operated with reduced tire pressure and one operated at high tire pressure, on parallel traffic lanes near Carson City, Nevada. Hodges reported a significant reduction in road damage in the low-tire pressure traffic lane. Since that time, similar reports have supported this concept. Considerations were therefore made to use CTI for solving rock quality problems on the Toncan timber sale. There was an initial reluctance to use low tire pressure because the poor performance problems of the rock were so extensive. It was difficult to imagine that simply lowering the tire pressure would prevent the 18-in. ruts that developed daily on the project.

The total contract cost was $2.5 million to construct 18.7 mi of road. A cost analysis indicated that the additional expense to import higher-quality materials would exceed $550,000. This amount was compared to $49,400 to equip the contractor's trucks with radial tires. Because of the wide difference, an 11 to 1 differential, it was decided to try the lower tire pressure alternative with the knowledge that some risks of failure would be assumed, because there was no previous testing at the time to indicate whether reduced tire pressure would work on poor-to-marginal road-surfacing materials. If this alternative did not solve the rock performance problem, the higher-cost alternative of importing higher-quality rock could be implemented.

FIGURE 6 Rock borrow after initial placement.

Significant differences in conditions and equipment existed on the Toncan project when compared to the earlier low-tire pressure studies. Virtually all the earlier reporting pertained to tubeless radial tires. Tubeless radial tires are available for standard-sized highway trucks, but were not available in the U.S. market for the loads and vehicles being used on the Toncan project. The contractor used four hauling vehicles, two Mack DM-800s and two Hayes trucks similar in size to the Mack DM-800. The gross weight is 85,000 lb with a 45,000-lb load capacity. The front axles were rated at a capacity of 20,000 lb and the rear at 65,000 lb. Scales were not available on this project, although on other projects the front axles normally weigh about 16,000 lb with 70,000 lb on the rear. On the basis of the load capacity of the Hayes and Mack dump trucks being used, 12.00R24 tube-type tires were needed to carry the loads, as shown in Figure 8. Because the haul route was all on a low-speed (less than 35 mph), low-volume, Forest Service road system, and CTI equipment was still in the developmental stages and was not readily available for immediate installation, it was decided to use a constant reduced tire pressure, rather than using CTI equipment. On the basis of the testing done by Hodges (*2*), tire pressures should be adjusted on the basis of the concept of percent deflection, or the ratio of the difference between unloaded and loaded tire section height to the unloaded section height. Accepting this concept, the optimum tire deflection to reduce road damage was reported to fall in the range of 20 to 22 percent. Conventional high tire pressure normally falls in the range of 10 to 12 percent deflection. Final tire pressures were set through a trial-and-error method on the basis of the loaded truck,

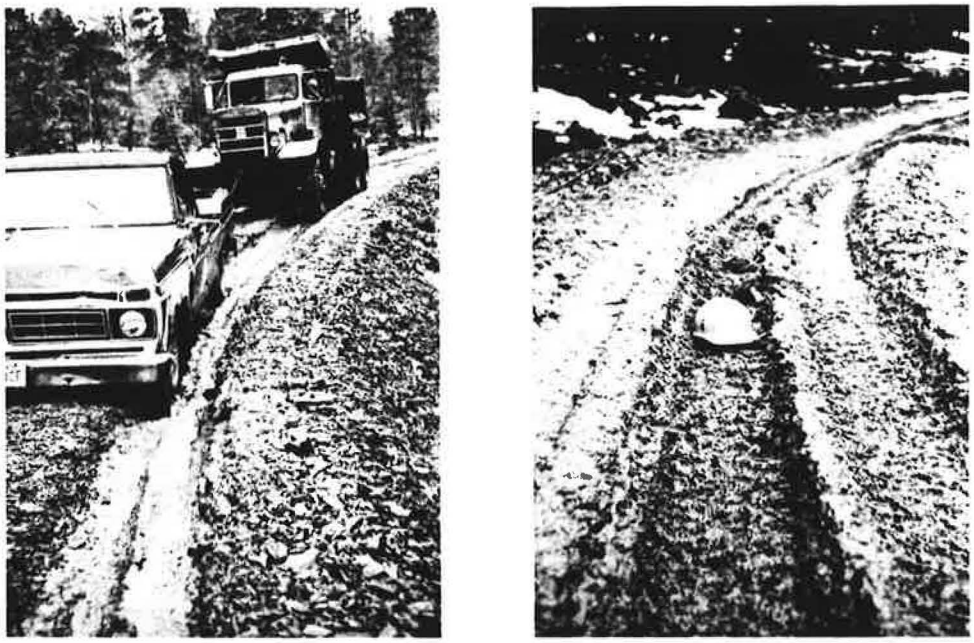

FIGURE 7 Deep ruts resulting from construction haul.

using a large caliper to measure tire section heights as shown in Figure 9. The final tire pressures varied because of the different load capacity of the trucks used on the project as well as the difference between front- and rear-axle loading. Final pressures selected ranged from 42 to 62 pounds per square inch (psi). Normal tire inflation pressures of 100 to 110 psi are used for the same tire and load.

RESULTS USING REDUCED TIRE PRESSURES

In late November 1987, rock haul resumed with the newly installed radial tires operated with reduced tire pressures as described earlier. To everyone's surprise, immediate results were realized. The deep rutting was virtually eliminated with the reduced tire pressure. The low-pressure radial tires acted similarly to pneumatic rollers and compacted the road surface, rather than producing deep agitation of the road base. Where road widths allowed, the drivers varied the wheel path of the construction vehicles that served to compact and seal the entire roadway, effectively preventing further moisture infiltration, as shown in Figure 10. In narrower road segments, typically on steep 10 to 15 percent grades, traffic was concentrated to a confined wheel path and some rutting did develop (see Figure 11). However, measured rut depth rarely exceeded 4 in. The road contractor was extremely pleased with the results and elected to equip his entire log haul fleet with radial tires to take advantage of the reduced road damage and maintenance.

Work continued on the Toncan project for several weeks until snow prevented further operations. No road maintenance was required during this late season of operations, a

FIGURE 8 Mounted 12.00R24 radial tires.

FIGURE 9 Bridge deck used for setting pressures and deflections.

FIGURE 10 Resulting road surface, full-width travel.

FIGURE 11 Maximum rutting where trucks could not vary wheel path.

big change from the almost continuous blade maintenance that was required with high-pressure tires. When haul commenced the following spring, the road again began to deteriorate and deep rutting started to develop. Frost in the road prism was suspected and a work shutdown was considered; however, a check of tire pressures revealed that one truck was operating with conventional high tire pressures (90 psi). When the pressures where reduced, the road immediately began to heal and continued to do so until construction was completed.

CONCLUSIONS

At present, the Forest Service spends a considerable amount of money to pay for importation of high-quality aggregates. On the basis of the results of the Toncan project, the use of radial tires operated at a reduced constant tire pressure may substantially reduce road damage and maintenance, at a relatively low cost, on roads built with poor- to marginal-quality rock. On the Toncan sale alone, it saved the Forest Service an estimated $450,000 and also reduced the amount of fine silty soil that could have reached high-valued fish streams. More savings potentially could have been realized if CTI systems had been available to reduce pressures even more on the return empty truck, but with additional up-front expense. Lower-quality materials could be used for road construction in other parts of the nation for low-volume, low-speed, road systems if low-pressure radial tires are used. This process could dramatically reduce road construction and maintenance costs in some parts of the United States. In Alaska, this process can be used successfully at many sites where only poor-quality materials are present, at considerable savings to the government.

When considering the use of this concept of reduced tire pressure and CTI, care must be exercised in the proper selection of tires and wheels. In some instances where haul includes operation on high speed state highways systems, CTI equipment is necessary. In order for successful application of CTI equipment, proper installation and maintenance is extremely important.

REFERENCES

1. L. Della-Moretta. *Proof of Concept Test for Central Tire Inflation*. U.S. Department of Agriculture, Forest Service, San Dimas Equipment Development Center, 1984, pp. 1–10.
2. H. Hodges. *Central Tire Inflation*. Nevada Automotive Test Center, 1987, pp. 1–82.

Effects of Tire Deflection on Rear-Axle Torque

ROBERT SIMONSON

Data were obtained in the study of an 18-wheel western-type log truck outfitted with central tire inflation (CTI) and carrying a highway load. Operating on grades of 20 and 25 percent, drive shaft torque was monitored to quantify stress levels on the drive train while varying tire deflection. Information is provided on the limits of additional traction to be gained through the use of tire deflection when operating on steep terrain—without excessive tire slip or vehicle maintenance. Torque levels for each grade and deflection combination are presented. A combination of steep topography and escalating road construction costs have led to the use of shorter, steeper roads. In order to reach landings of harvesting operations with a minimum road length, spur roads are often constructed in the 20 to 25 percent slope range. Haul roads exist with grades of 15 to 20 percent. An assist vehicle is often appraised when grades exceed 16 percent. Using the proper tire deflection for the application (based on speed and load) appears feasible through the use of CTI systems. Benefits have been seen in reduced vehicular damage to forest roads and increased tire life. An additional benefit realized with the use of CTI is improved traction on some road surface types because of the increased tire tread length.

A combination of steep topography and escalating road construction costs has led to the use of steeper roads. In reaching a needed elevation, steeper roads reduce the length of construction necessary. A road located along a ridgeline can significantly reduce resource impacts and excavation costs compared to a sidehill road.

In timber harvesting operations, spurs are often constructed in the 20 to 25 percent range to reach landings; with some in the Northwest containing pitches up to 30 percent (M. Rebar, unpublished data). Haul roads exist with grades of 15 to 20 percent. A cost allowance, and often a safety requirement is made to have a vehicle such as a road grader available to assist the log trucks in negotiating these steeper grades. The Forest Service routinely considers appraisal for an assist vehicle when grades exceed 16 percent (M. Rebar, unpublished data).

The U.S. Forest Service is evaluating the effects of tire deflection on tires, roads, and vehicles. Tire deflection is defined as the change in section height from the freestanding height to the loaded height (Figure 1). The percent deflection is the ratio of that change to the free standing section height ($\times 100$) (*1*).

Using the proper deflection for the application (based on speed and load) is feasible through the use of central tire inflation (CTI) systems.

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CTI is a system incorporated in a wheeled vehicle that permits the vehicle tire pressures to be regulated by the vehicle driver from within the vehicle cab while on the move. Operation of the system is simple. Air enters through the air cleaner and is compressed and dried. A priority switch ensures that sufficient pressure is available for the brake system before allowing air to be used for the inflation system. The controller in the cab operates either a deflate or inflate valve, exhausting air out of the system or increasing system pressure. The controller circuitry makes this decision on the basis of the selection of the driver and sensors monitoring vehicle speed and current system pressure.

Readily apparent benefits have been seen in reduced vehicular damage to forest roads (*2*), which has prompted nationwide use of the concepts in select trials. This is of particular interest on steep roads, because road maintenance costs escalate with increased grades (*3*). Some types of maintenance equipment cannot be used on the steepest grades because of safety concerns.

An additional benefit realized with the use of CTI is improved traction on some road surface types because of the increased tire tread length. A CTI trial was conducted in which loaded trucks climbed grades of 18 percent—and at times 21 percent—under their own power (assist vehicles were attached as required by State regulations) (*4*). Although it appeared operationally feasible to climb these steep grades using increased tire deflection without assistance, concern remained that the effects of possible added strain on the truck drive-axle was unknown.

OBJECTIVE AND SCOPE

The hypothesis of this project was that impact loads on the rear-axle gear train decrease with an increase in tire deflection on steep adverse grades.

Data were obtained in the study of an 18-wheel western type log truck outfitted with CTI and carrying a highway load. Operating on grades of 20 to 25 percent, drive shaft torque was monitored to quantify stress levels on the drive train while varying tire deflection. Torque levels for each grade and deflection combination are presented.

Wheel slip on these tests would be similar to that on any crushed aggregate with roughly the same gradation and density. Experience has shown that traction on the surfaces used in this study should be fairly consistent from optimal moisture to a saturated condition (R. Young, unpublished data). Somewhere between optimal moisture and zero percent moisture, traction would decrease.

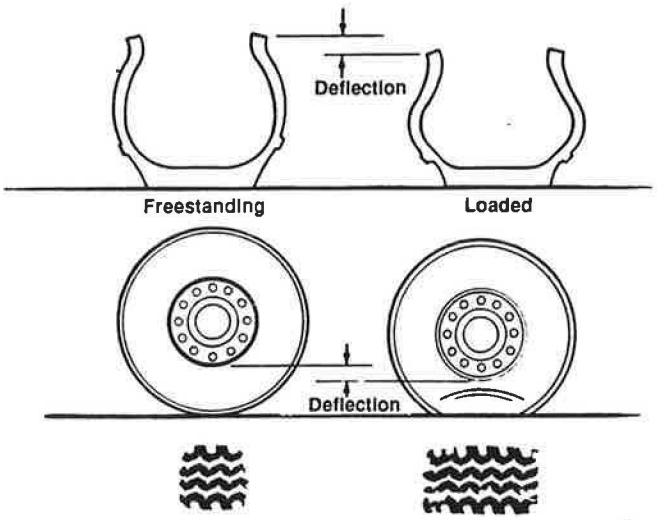

FIGURE 1 Tire deflection.

TORQUE

Axles are rated for a continuous torque on the highway (on the basis of a 3 percent grade, 60 mph, and rolling resistance from normally inflated tires on asphalt) and an intermittent climbing torque (not to exceed three times the continuous torque) (B. Hicks, unpublished data). For an 80,000-lb GCW truck, the design torque for continuous operation would be 10,250 ft-lb, the intermittent torques not exceeding 30,750 ft-lb.

An informal survey of truck distributors indicated that, in the Northwest logging industry, Eaton two-speed axles account for about 85 percent of the trucks purchased in 1989 (Dillon, Egge, Helms, and Spies, unpublished data). For this size logging vehicle, the Eaton DT440, or the DT461 tandem would be typical.

For a calculation of torque before testing, an approximation can be made based solely on the force required to overcome the weight of the vehicle. This simplification does not consider acceleration or rolling resistance. The vehicle weight in this test, of 75,600 lb acting vertically, may be broken into two components; a normal force, and a force F parallel to the road surface. With a road grade of 20 percent this force is

$$F = 75{,}600 * \sin[\arctan(0.20)] = 14{,}826 \text{ lb}$$

To generate the force F at the road surface with tires having a radii of 21 in. requires a total tandem axle torque of

$$T = 14{,}826 * 21/12 = 25{,}946 \text{ ft-lb}$$

STUDY SITE

The study site is the Mapleton District of the Siuslaw National Forest. Two roads were chosen by district engineers as typical—both surfaced with dense basalt, one with 1½-in. (−) material, and the other with 3-in. (−). This surfacing material is used on 90 percent of the roads with grades exceeding 16 percent on the Siuslaw National Forest. It is also the recommended material for steep grade construction throughout the Forest Service in the Northwest (H. Rickard, unpublished data).

The first series of tests was conducted on Billy Creek Road, which is typical of steep, maintained National Forest roads. The test grade here was 20 percent, and it was surfaced with a 1½-in. (−) material with 89 percent passing the half-inch sieve. Alignment was straight. Moisture content was 7.4 percent at the 2-in. depth. The road was in excellent shape before testing and no additional preparation was needed.

The second series of tests were on the Prong Spur, which was typical of a steep short spur with surfacing placed for temporary use. Its grade was 25 percent, and it was surfaced with a 3-in. (−) material with 59 percent passing the half-inch sieve. Moisture content averaged 7.6 percent at the 2-in. depth. This road was graded to remove material which had recently sloughed off of the cut bank onto the roadway.

Each section was tested once at the beginning of the project for gradation, Atterberg Limits, the Los Angeles Abrasion, Durability Index, and Sand Equivalent, since truck tests ran for only one week. Testing was performed in a materials laboratory on a representative sample.

Each test section was sampled for aggregate density and moisture content. Values were obtained by taking nuclear densometer readings in each of three randomly selected locations and averaging for each section.

TEST EQUIPMENT

Although the test vehicle had axles and a drive line of lighter duty than those commonly used in the industry, the torques required to propel any log truck with the same approximate loaded weight distribution would be similar.

The vehicle tested was a Diamond REO with tandem axles and a wheel base of 207 in. It was originally a 10-yard dump truck that was converted by Page Equipment to a conventional logging truck. Because the engine size, transmission, drive axle unit and tire size remained unchanged after the conversion, the tractive effort capability remained unchanged. It does not change when the truck is coupled with various trailers and loading (5).

The test vehicle was highway loaded with logs to 75,600 lb GCW for testing. A GCW of 76,000 lb is average for the Mapleton area (D. Upton, unpublished data); 74,805 lb GCW was the average log truck weight in the Oregon Coast range as reported by Stryker (6). The load distribution of the test vehicle was 10,880 lb on the steer axle, 33,540 lb on the drivers, and 31,180 lb on the trailer.

The truck is powered by a 350-hp engine and the drive train consists of an Allison five-speed automatic transmission coupled by drive shafts to Eaton 34DT two-speed axles. The interaxle differential lockout remained engaged throughout the testing. The low-end differential ratio is 7.60:1. The truck is equipped with Michelin XZY 11R24.5 tubeless tires.

The truck was instrumented with 11 transducers to measure

1. Drive shaft stress;
2. Left walking beam load, forward;

3. Left walking beam load, aft;
4. Front axle load;
5. RPM, left front wheel;
6. RPM, left front driver;
7. RPM, left rear driver;
8. Pressure, steer tires;
9. Pressure, drive tires;
10. Inclinometer, and
11. RPM, drive shaft.

The drive shaft was chosen as the most easily instrumented link in the drive train assembly. Drive shaft stress was measured using gauges installed on a clear section of the drive shaft with orientation for torque sensitivity only. Output signals from the gauge bridge were wired to an FM transmitter. The FM signals were then wired to a shaft-mounted antenna. The drive shaft was calibrated off the truck on an assembly using a hydraulic ram to provide known torque loads.

STUDY METHODS

To test the hypothesis, tests were conducted by driving the same loaded logging truck over test sections on each of the test roads, using tire deflections of 10, 15, and 20 percent while monitoring the strain (torque) on the drive train. Tires were run at constant deflections rather than constant inflation pressures because it is desirable to keep the spring and damping rate of the tires constant, thus keeping the dynamic load experienced by the truck and road constant (*7*). Ten percent is the typical tire deflection of a loaded 18-wheel truck operating in the 90- to 100-psi tire inflation range common today. Twenty percent is the deflection the Forest Service is advocating for use by vehicles traveling at low speed on its low-standard unpaved roads and which has been approved by the Tire and Rim Association (*8,9*) for use in Forest Service trials. Fifteen percent was chosen as an intermediate point to test, because trucks inflate and deflate on the go and are therefore often operating in the range of 10 to 20 percent rather than at one or the other.

In a typical CTI-equipped truck, tire pressure settings to attain a desired deflection would be programmed in the control unit on the basis of that truck's typical load distribution. Because loads vary by a few thousand pounds, actual deflection on a given haul would only approximate the desired deflection. On this test, the actual deflection was measured and set for the exact test load.

Also in a typical CTI installation, there are two discrete pressure channels; one for the steer tires, another for the remaining 16 tires. Because no log load can be perfectly balanced, individual tire loads, and therefore individual tire deflections, vary because pressure from tire to tire within the same channel is the same. In a field sampling of load distributions, it is not uncommon to find axle end loads varying by 40 percent on the same truck (*10*). Individual axle end loads were not measured for this test. Measured deflection was averaged within each CTI control channel to attain the test deflections.

On this particular combination of truck, load, and tire, the cold pressures used for the desired deflections were as follows:

Deflection (%)	*Steer Tires (psi)*	*Drive/Trailer Tires (psi)*
10	–	99
15	99	76
20	59	45

A pressure of 99 psi was the highest pressure obtainable with the particular model of CTI onboard the truck, therefore 10 percent deflection was not attainable on the steer tires.

Prior study of washboarding indicated that tire slip on logging vehicles occurred in the range of 12 Hz (L. Della-Moretta, unpublished data), therefore a sampling frequency of 15 Hz was chosen as sufficient to capture this phenomenon. It was decided that 20 samples were sufficient to capture an occurrence of interest, so the instrumentation sampled at a rate of 300 samples per second.

A number of ground rules were followed to maintain continuity of the analysis of the data gathered during specific tests.

The ground rules are as follows:

- All tests were conducted with the same driver to minimize driver variation.
- All tests were conducted when wind conditions were below 5 mph to keep the effect of wind resistance on the truck to a negligible level.
- Tests were conducted when precipitation was negligible (less than 0.03 in. per day) to avoid saturated road conditions.
- Preparation for all planned tests was preceded by a full checkout of all hardware/equipment.
- Before and after each test series, the instrumentation was turned on and data acquired for a period of 5 to 10 sec to assist in the validity of data analysis.
- Each test condition was repeated a minimum of four times.
- Tests were run in a sequence of decreasing deflection on the 25 percent adverse grade. This was because with decreased deflection, the truck lost traction sooner, and would not reach or be affected by the disturbed road surface resulting from the previous tests' traction loss. The sequence was reversed on the 20 percent adverse grade to verify that trends detected in the tests were indeed based on deflection, and not testing sequence.
- Tests were conducted as close as feasible to a steady 3 mph. This minimized the effects of wind resistance and acceleration.

An assist grader was on site at all times for safety, but was not used during test runs.

SLIP AND TORQUE

Vehicles propel themselves by transferring the energy developed by the engine quite efficiently through the gear train to force on the rotating tire. The transmission of this force is dependent on the driven tire loading and the coefficient of traction between the tire and road surface (*11*). However, at the contact of the tire to an unpaved road only a percentage of this energy is transferred to movement; much of it is lost in tire slip and soil shear. This slip excites the road/tire/suspension system and washboarding is a result (*12*).

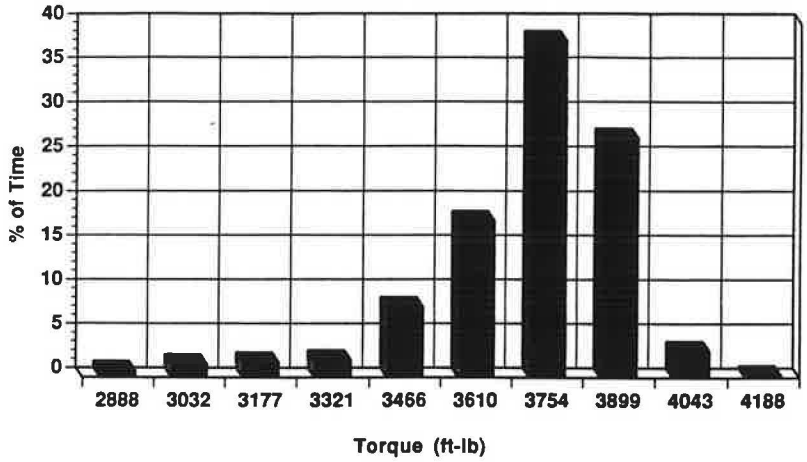

FIGURE 2 Drive shaft torque 10 percent adverse, 10 percent deflection.

On a given road surface with a given tire, CTI introduces the ability to increase the tire-soil contact area reducing the unit shear stress seen by the road surface. When the road surface is able to withstand this reduced shear, less energy is lost to slip and less road damage occurs. At the same time, the damping of any cyclic bearing loads is increased by the softening of the sidewall and lengthening of the tire contact patch. These factors are the main premises behind promoting the use of CTI systems on unpaved forest roads.

On steeper roads, the torque necessary to be transferred by the tires into forward movement is increased because of the increasing effects of gravity. The additional torque required for this task is not usually the limiting factor in today's vehicles, but the failure (i.e., tire slip and soil shear) at the tire-soil interfaces.

Some slip is inherent in the design of today's elastic pneumatic tires. Even before the visual failure of a spinout, the tire slips, the road surface shears, and consequently the tire sinks deeper. This sinkage continues causing increased contact area until the slip stops. As the tire is slipping, the tire is rotating faster and less torque is being transmitted. When the slip stops, the tire grabs and the torque requirement to continue moving forward is instantaneously increased.

As tire deflection and the contact area is increased, soil shear is reduced. When the tire does slip, the sinkage required to stop the slip, and therefore the elapsed time slipping, is also reduced. The rotational velocity attained by the slipping tire is less because there is less elapsed time for it to accelerate, and so the instantaneous peak torque required to arrest the spinning tire is also reduced. This concept was demonstrated in the field testing and is illustrated in the following charts of torque distributions as measured on the test vehicle.

TEST RESULTS

Drive shaft torque for each tire pressure on each grade is plotted in a histogram (Figures 2–7). These histograms provide a simple visual method of discerning the difference in peak and average torques for given conditions. Note that each stack in the histograms is labeled on the horizontal axis with the low torque of each interval. The vertical axis shows the percent of time on the constant grade that drive shaft was operating within the given torque range. The sum of the stacks in each histogram is 100 percent.

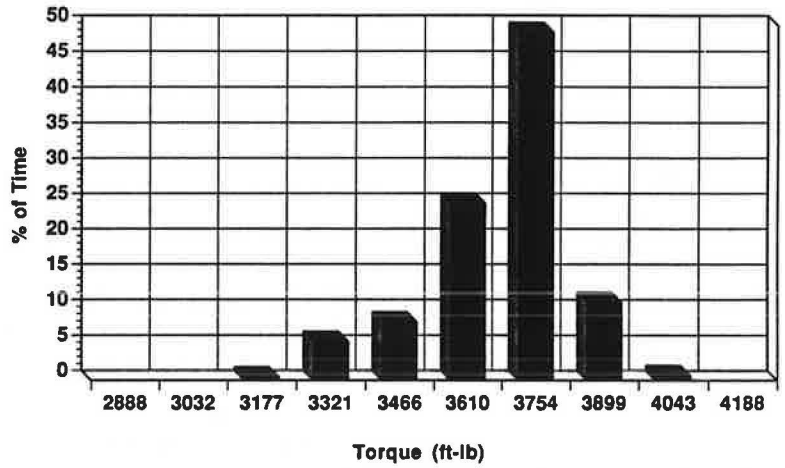

FIGURE 3 Drive shaft torque 10 percent adverse, 15 percent deflection.

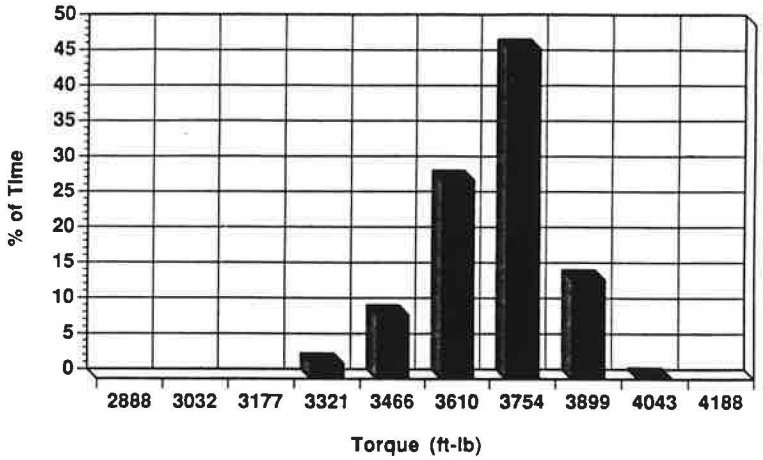

FIGURE 4 Drive shaft torque 20 percent adverse, 20 percent deflection.

It can be seen in Figure 2, with 10 percent tire deflection, the distribution of torques is wider than in Figure 4, with 20 percent deflection. Peak torques are higher at lesser deflection, and because the tire is slipping more often the torque drops more often.

Before testing, it was expected that the average torque would increase with greater deflection, because of the additional work used in flexing the tire on the compacted test sections. Rolling resistance is the sum of work to compress and deflect the road surface, flex the tire, overcome rolling friction (as in the bearings), and to overcome air frictions about the tire. The average drive shaft torque required to climb the grade did not significantly change, ranging from a low of 3,766 ft-lb at 15 percent tire deflection to a high of 3,780 ft-lb at 10 percent tire deflection (3,775 ft-lb at 20 percent). This would indicate that the additional rolling resistance of the tire expected with greater tire deflection is not a detectable amount, or is negated by the reduced work going into deflecting the road surface. The character of the road surface, where measurements were recorded, did not visibly change during the testing.

Operating the test vehicle on a 25 percent adverse grade demonstrates the results once again as shown in Figures 5–7. Peak torques are higher at lesser tire deflection, and the distribution of torques is wider. In Figure 5, besides a peak in the distribution of torques about the average, a lesser peak of values occurs at the 3,047 to 3,482 ft-lb range. This agrees with visual observations of the test vehicle experiencing significant additional slip at the 10 percent tire deflection on this steep grade.

Here again the average torques do not seem to reflect any logical pattern from the expected increase in rolling resistance caused by additional tire deflection. Average torque required to climb the 25 percent grade was 4,770 ft-lb.

CONCLUSIONS

With the gear ratio of 7.60:1 in the test vehicle differential, the average torque in the drive axles is 7.6 times that measured in the drive shaft less the loss to the inefficiency of the rear end. Drive train efficiency (Dte) is estimated by Dte =

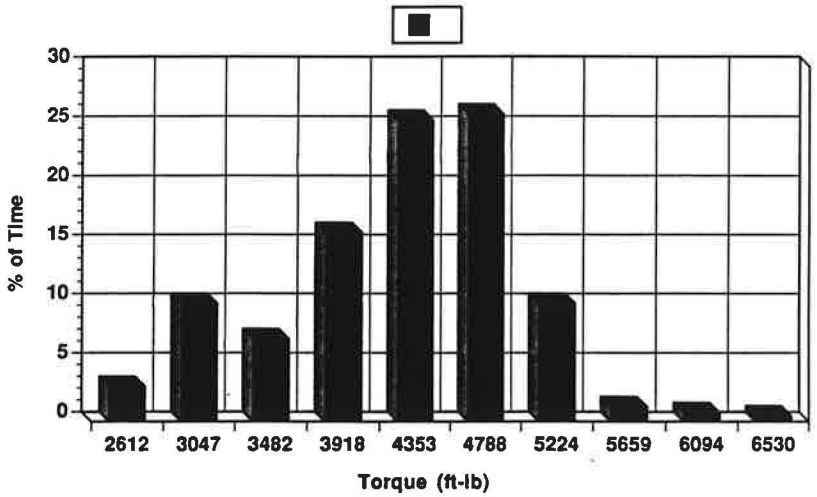

FIGURE 5 Drive shaft torque 25 percent adverse, 10 percent deflection.

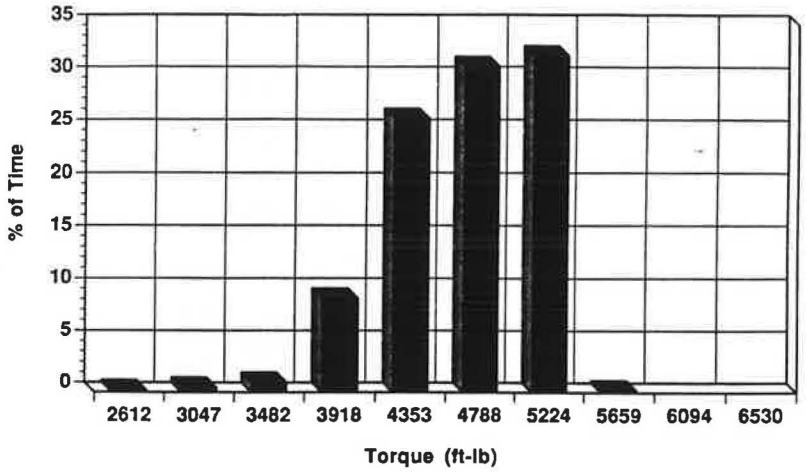

FIGURE 6 Drive shaft torque 25 percent adverse, 15 percent deflection.

$(1 - 0.05N)$, where N is the number of components in the drive train (*13*). In this case, torque was measured in the drive shaft and only one component, the rear end transmission, lies between the shaft and the drive axle. Efficiency is then estimated at 95 percent.

Average drive shaft torques measured to pull the 20 and 25 percent grades were 3,775 and 4,770 ft-lb, respectively. These amounts yield drive axle torques in the range of 27,200 and 34,400 ft-lb.

Tire deflection did not affect the most practical consideration of traction, gradeability. The test vehicle, operated at a slow and steady speed, climbed the 20 percent grade independent of tire deflection. On the 25 percent grade, the vehicle could not sustain the climb, again independent of tire deflection. The greater the tire deflection, the farther the vehicle traveled before totally losing its traction. Most likely, the greater tire deflection required a slightly larger bump, or a slightly weaker spot in the road surface before finally losing traction. This slight difference in tractive capability on a packed road surface, as is typical in the Northwest, is not sufficient reason on its own to operate at any particular deflection.

These test roads had near-ideal moisture contents and compactions. On unsurfaced spurs and landings, the difference in tractive capability would be expected to be greater, as seen in other testing (*14*).

At increased tire deflection, the data do point to less tire slip, fewer peak torques (instantaneous tire grabbing), and therefore more torque efficiently applied to moving the vehicle up the grade. Even in the situation on the 25 percent grade in which an assist vehicle would still be necessary, increased tire deflection would translate the power of the engine where it is needed rather than wasting it in road damage (tire slip) and additional gear train wear (torque spikes).

On the subject of drive train life, there are two factors to consider. Gear bending stresses vary linearly with torque. Contact stresses that include gear pitting and bearing fatigue are logarithmically dependent on stress (the square root of torque) (B. Hicks, unpublished data).

Average torque was not affected by tire deflection yet peak torques were reduced 14 percent while operating on a sustained 20 percent adverse grade, and reduced 9.6 percent on the 25 percent adverse. These results were accomplished by

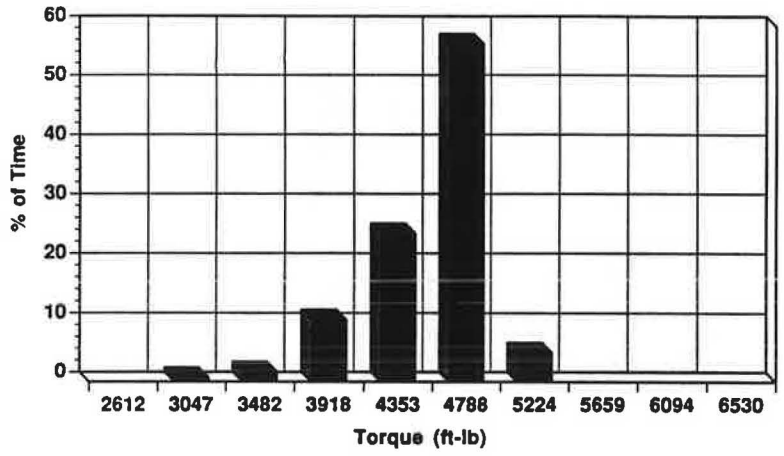

FIGURE 7 Drive shaft torque 25 percent adverse, 20 percent deflection.

increasing tire deflection from the typical 10 percent to the 20 percent approved in the interim standard issued by the Tire and Rim Association (*8,9*).

Peak axle torques on the 20 percent adverse grade ranged from 29,300 to 29,500 ft-lb at 15 and 20 percent deflection, respectively. Operating at the common tire deflection of 10 percent produced peak torques up to 34,200 ft-lb. Climbing the 25 percent grade produced peaks in the 38,500 to 42,500 ft-lb range.

Increasing tire deflection then does not have an adverse impact on drive train life on steep grades. The results of this test would indicate that it may increase the mean time to failure.

ACKNOWLEDGMENTS

The author would like to acknowledge the assistance of the San Dimas shop crew and technician support, which put in many long hours; the road crew and engineering staff of the Siuslaw National Forest, Mapleton District, who jumped to assist, often late at night or on their days off; and a special thanks to LaMoure Besse for his continual assistance through all phases of this test.

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Truck Operation at Constant Reduced Tire Pressure

GREG WATKINS

The U.S. Forest Service, in cooperation with log truck operators and tire manufacturers, has been operating loaded trucks with tire pressures down to 65 psi. This pressure is significantly lower than the 90- to 110-psi tire pressures normally used by the trucking industry. The tires remain at this constant but reduced pressure and the pressure is not increased when the truck is operated on paved roads at highway speeds. The pressure selected is the lowest allowable pressure considering the maximum load and speed the vehicle will encounter during its operation. In this study, 65-psi tire pressures were used and maximum speed on paved highways was restricted to 55 mph. Operation at constant reduced tire pressure can accomplish many of the benefits obtained with central tire inflation (CTI) systems, but without the need for expensive hardware. No detrimental effects were observed in the tire casings, nor was there an increase in fuel consumption. Benefits include reduced road damage to roads with weak structural sections. Ride quality and traction also improved. Limitations do apply to the length of the loaded travel at highway speeds. Vehicles must be equipped with radial tires. This study at constant reduced tire pressure is an outgrowth of CTI studies that vary tire pressure to suit the load, road surface, and speed of the vehicle.

The USDA Forest Service, in cooperation with log truck operators and tire manufacturers, has been operating log trucks with tire pressures significantly lower than the pressures normally used by the trucking industry. In 1984, the Forest Service began testing the use of lower tire pressures in conjunction with central tire inflation (CTI) studies. The basic principle of CTI is to vary tire pressures to select the optimum tire pressure for each phase of a particular vehicle operation.

This tire operation at constant reduced low pressures differs from CTI in that the pressure in the tires is not varied. It is set at a constant pressure, on the basis of the maximum speed and load for the specific hauling operation. The pressures selected are within the tire manufacturers' recommended practices, but are significantly lower than the pressures customarily used by truckers. The tire pressure selected is based on the condition of the road surface and the maximum speed and load the tires will encounter. In this study, 65-psi tire pressures were used and maximum speeds on the paved highways were restricted to 55 mph.

BACKGROUND

It was documented in the 1987 Low-Volume Road Proceedings, that reduced tire pressures are beneficial both to paved and unpaved roads (*1*). Vehicle mobility, ride, fuel economy, and road surface conditions all improved when low tire pressures were used on unpaved roads. Tire pressures were then increased for vehicle operation on paved highways at higher speeds. Loaded log trucks have historically operated with tire pressures between 90 and 110 psi.

Heavy vehicles cause a disproportionate amount of damage to the structural sections of roads. Damage may occur regardless of whether the road has native, gravel, or asphalt surfacing. A reduction in tire pressure distributes the tire load to a larger area of the road's structural section, thus reducing unit loadings.

Many truck haul operations involve a portion of travel on unpaved roads and the remainder on paved highways. CTI systems are ideal for adjusting tire pressure to match the load and speed requirements for tires. However, the cost and limited availability of the hardware needed to change tire pressures from the truck cab are disadvantages. Stationary airing stations may be used to vary the pressure in tires, but they also require expensive hardware and create costly delays for the trucker.

A simpler method was needed to achieve many of the benefits that reduced tire pressures create for weak or unpaved roadbeds and still permit the flexibility to operate on paved highways at higher speeds.

Rationale for Selecting Tire Pressures

The limiting factor for this study was the selection of a reduced tire pressure that would benefit unpaved roads and would still permit loaded haul at highway speeds. Sixty-five pounds per square inch for the drive and trailer tire pressures was selected for the following reasons:

- A CTI demonstration project on the Mendocino National Forest near Ukiah, California, used air compressors to vary tire pressures. Loaded trucks operating with 45-psi tires produced beneficial results to the unpaved road. In another project, Foglio Trucking, on the Siuslaw National Forest in western Oregon, operated CTI-equipped trucks with 60-psi tires. By visual observation, the aggregate surfaced roads in both projects experienced similar beneficial results with 45- and 60-psi tires.
- A drawbar pull test in Auburn, Alabama, studied tractive effort as related to tire pressure (*2*). Loaded trucks were tested with tires inflated to 30, 65, and 100 psi. Test results showed a 34 percent increase in pull on a sandy soil and a 17 percent increase on wet clay for tires at 65 psi over tires at 100 psi.

USDA Forest Service, 2245 Morello Ave., Pleasant Hill, Calif. 94523.

There was not a significant increase in drawbar pull on either road surface when tire pressures were decreased from 65 to 30 psi.

- These findings suggested that tire pressures near 60 psi would provide significant benefits to unpaved roads. The pressure of 65 psi was selected for the test project because it was the lowest pressure that would still permit 55-mph haul on highways according to tire manufacturers' requirements (see Table 1).

Tables for Tire Pressure, Load, and Speed

The National Tire and Rim Association and individual tire manufacturers publish information regarding minimum and maximum tire pressures for various tire loads and speeds. This test of operation at constant reduced tire pressure met manufacturers' recommended pressures for the specific tire used. Table 1 was developed from Michelin's truck tire data book for 11R24.5 highway tires (*3*).

For this test, Michelin authorized a 4 percent increase in load with a speed restriction of 55 mph. Each dual tire with 65 psi carried 4,250 lb. Additional increases in load are permitted with each 5-mph reduction speed.

CASE STUDIES

Three separate studies using constant reduced tire pressures were conducted: the Quincy, Soper-Wheeler, and Sonora tests. The Quincy test consisted of six log trucks equipped with new Michelin 11R24.5 radial truck tires on all 18 wheels. Tire inflation pressure was set at 65 psi in the eight drive tires and in the eight trailer tires. The steering tires were set 10 psi higher, at 75 psi, to provide better steering response. When loaded, the trucks carried up to 80,000 GVW. These reduced tire pressures still permitted operation on highways at speeds up to 55 mph when loaded.

Figure 1 displays the axle configuration and tire pressures for the Quincy and Sonora tests. The Soper-Wheeler test used pressures 5 psi higher.

The Quincy test began in the spring of 1988 and has operated for three 8-month logging seasons. Clover Logging's trucks hauled logs from over a dozen different locations on the Plumas National Forest to the lumber mill in Quincy, California. The haul route typically included a native or gravel surfaced forest road, a paved or chip sealed county road, and a paved state highway. The duration of haul on the paved highways

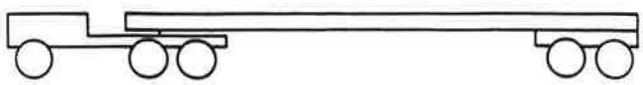

FIGURE 1 Axle configuration and tire pressures for the Quincy and Sonora tests.

was less than 1 hr. Speeds were restricted to a maximum of 55 mph. The typical round-trip haul was 50 mi and consisted of approximately equal distance on paved roads and on gravel or native-surfaced roads.

The Soper-Wheeler test was conducted in the spring and summer of 1990. Soper-Wheeler, Inc., a timber company near Oroville, California, operated four trucks at constant reduced pressures. Trucks used existing 11R24.5 radial tires manufactured by Dunlop. Pressures were deflated from their customary 100+ to 70 psi. Steering axle tires were again 10 psi higher, at 80 psi. The haul route consisted of approximately 20 mi of unpaved forest roads, 25 mi of paved county roads, and 60 mi of straight, level state highway. The determination of the effects of lower tire pressures on fuel consumption was a primary objective of the test because the travel was mostly at higher speeds on paved highways.

The Sonora test was the only test to operate during the winter. The haul route included 15 mi of aggregate and native surfaced forest and county roads and 25 mi of paved county and state highways near Sonora, California. Six log trucks, owned by Lone Pine Logging, were operated at the tire pressures shown in Figure 1. The road typically experienced morning freezing of snow or rain water and roadbed thawing in the afternoon. The test was operated partly because of the unacceptable condition of this road the previous winter while using high tire pressures.

RESULTS

Road Surface

The unpaved roads in the Quincy test required significantly less maintenance blading than they did before the use of reduced tire pressure operations. One gravel road, for example, required only one spot blading instead of the customary three complete bladings for the same volume of traffic. The benefits to the unpaved road were similar to those experienced in other variable tire pressure operations in which tire pressures were

TABLE 1 RECOMMENDED TIRE PRESSURES FOR VARIOUS TIRE LOADS AND SPEEDS (*3*)

	Speeds up to 65 mph							
psi	65	70	75	80	85	90	95	100
Single tire	4,465	4,705	4,960	5,235	5,513	5,780	6,105	6,430
Dual tires	4,079	4,285	4,534	4,740	4,976	5,200	5,495	5,790
Speeds restricted to 55 mph								
Single tire	4,559	4,799	5,059	5,340	5,623	5,896	6,227	6,559
Dual tires	4,160	4,371	4,625	4,834	5,076	5,304	5,605	6,690

varied with the use of air compressor stations or with CTI systems (*3*).

The Soper-Wheeler test also experienced a noticeable reduction in washboarding of the unpaved roads.

The Sonora test created a marked improvement in the condition of the unpaved county road. This road had become almost impassable during the previous winter log haul operation using high tire pressures. Local residents living along the road had complained of difficulty in driving the road because of segments of deeply rutted road. There was a significant improvement in the surface condition of the road the winter in which reduced tire pressures were operated. The deep rutting of the road experienced with the high tire pressures the previous year did not occur. There were similar amounts of rainfall both winters. It was the consensus of road users that the road surface remained in good condition during the entire winter season of haul.

Ride quality for the truck operator, especially in the loaded truck, improved in all three tests. The increase in vehicle traction was especially apparent to drivers on the slippery road in the Sonora test. Other CTI studies (*4*) have indicated benefits to paved structural section in addition to unpaved road surfaces.

Fuel Consumption

During the second season of the Quincy test, four trucks were operated for 2 weeks at reduced tire pressure, then a week at high tire pressure of 90 psi. Fuel consumption and mileage traveled were recorded daily. Loaded truck weights were not recorded. Drivers used load scales on the log bunks to achieve as near as possible a maximum legal load for each trip. Three trucks obtained a 3 percent increase in fuel economy with high-pressure tires. One truck had a 4 percent increase with the reduced-pressure tires. Fuel consumption for all the trucks was in the range of 3.5 mi/gal. Considering variables of load and driver differences, no significant change in fuel consumption was noted.

One truck and driver in the Soper-Wheeler test was used to study the effects of fuel consumption. Logs were hauled from the Plumas National Forest to a saw mill in Paskenta, California. The log trailer was piggy-backed on the return trip. Seven round-trip loads were hauled a total of 1,462 mi with 90 psi in the tires. Fuel consumption was 4.75 mi/gal. Eight loads were hauled to the same destination using the same truck and driver, but with 70 psi in the tires. The lower tire pressure haul covered 1,711 mi and averaged 4.80 mi/gal. Fuel consumption with lower tire pressures showed a slight improvement of 0.05 mi/gal even though the haul was predominately on the highway at 55 mph.

Tire Wear

Specific tire tread life data were not a part of the study. The tires in the Quincy test have begun to be recapped. Recapping is generally occurring between 25,000 and 30,000 mi. The subjective evaluation of the truck owner is that tire wear is unchanged from the previous high tire pressure operations. Four tires were damaged during the test, but none of the damage was attributed to reduced tire pressure operation. The tires will continue to be operated at 65 psi for the life of the casing.

Tread Type

Michelin's XDHT tread pattern was used in the Quincy test. This is a cross-tread pattern creating individual blocks of tread. Drivers complained of a squirmish feel on the paved highway with the reduced tire pressures. Once the tread was half-worn, drivers reported a better feel to the tire. The XZY rib tread was used in another CTI test without this complaint. Rib treads are inherently more rigid and better suited for operation on paved roads at reduced pressures. Figure 2 shows the differences between the two tread patterns.

LIMITATIONS

Drive tires with 65 psi still constitute a relatively high tire pressure when the truck is empty. Drivers of empty trucks did experience some rough ride. With the trailer piggy-backed, the load per drive tire is less than 3,000 lb/tire. In the Quincy test, empty trucks did create some road washboarding on steep switchback curves on one of the haul roads. However, the washboarding was significantly less than occurred in previous years when 90 psi was the standard tire pressure. Previous CTI studies have shown that when the air pressure in the drive tires of the empty trucks is reduced to 25 psi, washboarding does not occur (*1,2*).

Trucks must be equipped with tubeless radial tires for operation at lower pressures. Travel at highway speeds may need to be limited because tire casing heat buildup could be detrimental on a sustained high-speed haul.

The length and maximum speed of the loaded haul on the highway affects the selection of the lowest permitted air pressure. Most tire manufacturers publish load tables for tire pressures down to 65 or 70 psi. These tables should be used in the selection of a specific tire pressure.

NEED FOR FUTURE TESTING

Additional testing is needed to determine if there is a quantifiable difference in the condition of a gravel road surface

FIGURE 2 Tread patterns of the XDHT cross rib tread (*left*) and the XZY rib tread (*right*).

when 65- and 70-psi tires are operated. The test should ideally study the effects of tire pressures between 45 and 90 psi. Additional testing is also needed to test road performance with unloaded trucks with a range of tire pressures down to 25 psi.

CONCLUSIONS

Operation at constant reduced tire pressure promises significant benefits for many truckers. It is simple and inexpensive to implement. When truck tire pressures were decreased from 90 psi to either 65 or 70 psi, the following conditions were noted:

- Unpaved road surfaces benefited significantly,
- Truck traction and ride improved,
- Fuel consumption was unchanged, and
- No detrimental effects were noted on tires.

This concept is applicable to log trucks, garbage trucks, ready-mix trucks, or buses. Any vehicle involved in haul situations involving unpaved roads or with a mix of paved and unpaved roads may benefit from operations at constant reduced tire pressure.

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Effects of Variable Tire Pressure on Tire Life

PAUL H. GREENFIELD AND ALAN E. COHN

Results were obtained from tests to determine the effects of variable tire pressure on hauling-vehicle tires. Tests address two major categories—tire wear rates and tire carcass life. Results indicate that the larger ground contact area of reduced tire pressure operation, in an off-highway condition, is not detrimental to tire wear, and may be beneficial to tire wear on rough roads where energy is wasted because of a vehicle's bounce or hop. Testing performed to date indicates no detrimental effects on tire carcass life, but it is deemed too early to make conclusive statements on the basis of the tests presented.

Since 1983, the U.S. Department of Agriculture Forest Service has been studying central tire inflation (CTI) as a means of reducing road costs. CTI is a system that enables the driver of a vehicle to control tire pressure from inside the cab while in motion. By matching the tire pressure to road, speed, and load conditions, less damage occurs to existing roads, and less surfacing material is required to support a particular volume of traffic. Testing indicates that significant benefits to low-volume roads can be realized by the use of CTI. Of major interest to the Forest Service is log-hauling vehicles and their impacts on roads. Consequently, most CTI efforts have been directed to studying the effects of reduced tire pressure operation with logging trucks.

One of the primary questions that surfaced early in this program was, "What effects will the reduced tire pressure have on tire performance?" It was clear that unless tires could perform satisfactorily, CTI would have no practical use in reducing costs associated with road construction and road maintenance. Additionally, tire manufacturers have concerns about warrantees on their tires being operated in reduced pressure service. The economics of reduced road costs as a result of using CTI is not addressed, but the effects of CTI on tire wear and the testing that has been performed to validate reduced pressure operation are discussed.

BACKGROUND

There are two major types of tires available for today's vehicles. These are bias-ply and radial tires. Because of the flexible sidewall design, only radial tires should be used for CTI applications.

Traditionally, the load-carrying capacity of all tires has been related to the enclosed air volume and pressure maintained in the tire carcass. In reality, the load-carrying capacity of radial tires is largely related to the tires' structural ability to support the loads through a system of tensile cords that extend from the bead of the tire (near the rim) to the *upper* outside portion of the tire. The current CTI test program addresses the load-carrying capacity of reduced-pressure tires running at restricted speeds.

To understand the principles of operation at reduced tire pressure, it is necessary to understand the concept of tire deflection. Tire deflection is defined as the change in section height from the freestanding height to the loaded height. The percent deflection is the ratio of that change to the freestanding section height times 100 (Figure 1).

Because the goal of a CTI system is to match tire pressure to road, speed, and load conditions, different pressure settings are needed as these conditions change. Typical systems installed today for standard on-highway logging trucks operate at three different pressure settings, on-highway, off-highway loaded, and off-highway empty. Forest Service research conducted by the Southern Forest Experiment Station located in Alabama (*1*), indicated that significant reductions in off-highway road damage occurred as tire deflection approached 20 percent. There was only a minor decrease in road damage as the deflection was increased to 30 percent. Because of this, the goal for off-highway, low-speed operation is to approach a 20 percent tire deflection and avoid the unknown effects of heat build-up with the greater deflection. This 20 percent tire deflection spreads the load of the vehicle over a larger tire footprint. For typical U.S. logging truck tires, this increase is about 60 percent greater in length. Higher-speed highway operation typically averages around 10 percent deflection.

With a goal of achieving a 20 percent tire deflection, tire pressures on different vehicles can vary widely because of a number of factors. These factors include the vehicle's load, axle distribution, and number of tires per axle. For a typical western U.S. logging truck carrying highway-legal loads, tire pressures for highway conditions would be 100 psi in the steers, drives, and trail tires. For an off-highway loaded condition, there would be about 90 psi in the steers and 54 psi in the drives and trailer tires. For an off-highway empty condition, about 80 psi in the steers and 25 psi in the drives and trailer tires would be expected. These settings would be for off-highway operation at speeds under 35 mph. Higher off-highway speeds would require slightly higher pressures to avoid the effects of heat build-up, and have to be set by the individual tire companies. Obviously, there is no CTI restriction on highway speeds.

The U.S. Tire and Rim Association sets tire pressure limits for load and speed for all tire manufacturers conducting busi-

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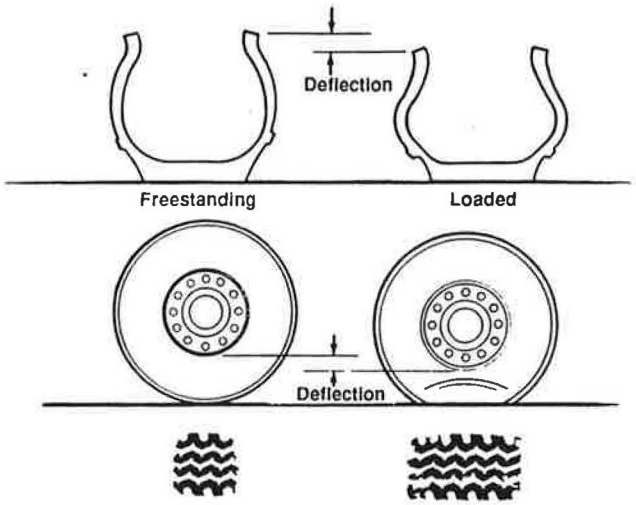


FIGURE 1 Tire deflection.

FIGURE 2 Photograph of a test truck.

ness in the United States. Individual manufacturers can provide additional pressure reductions in accordance with their own criteria, but only by permission of the manufacturer on a case-by-case basis. The Forest Service has obtained interim standards for reduced pressure operation from the U.S. Tire and Rim Association.

TESTING GOALS

The Forest Service has been working with various tire companies, the industry's Tire and Rim Association, and independent testing contractors to determine what effect reductions in tire pressure might have on tire performance. Testing of reduced pressure effects on tires covers two general categories:

1. Tire Tread Wear. These studies examine the effects of reduced tire pressure on rate of tread loss.
2. Tire Carcass Life. These studies examine the effects of reduced tire pressure on the structural performance of the tire. Because multiple retreading of tires is commonplace in the timber industry, the ability to retread a tire several times is being studied under this category.

The following sections cover specific testing procedures and results.

NEVADA AUTOMOTIVE TEST CENTER STRUCTURED TEST

Background

In May 1987, a structured test was conducted by the Nevada Automotive Test Center (NATC) for the Forest Service (*2*). Two identical 18-wheel log trucks were operated on parallel lanes over a closed-loop test course constructed to AASHTO specifications. One truck operated with standard highway tire pressure, the other with reduced tire pressure (Figure 2).

The test course consisted of 12 different roadway sections. These sections included a double penetration chip seal, asphalt concrete, an aggregate section with man-made potholes, and a severe rock course. The severe rock course consisted of implanted rocks, 4 to 6 in. in height with 2.25 in. of contact area. Each pass of the test vehicles over this section resulted in about 35 contacts being made with each rock on a particular tire. There were 520 passes made over the severe rock course, which equates to about 18,200 rock impacts per tire.

Testing Performed

The following tire tests were conducted:

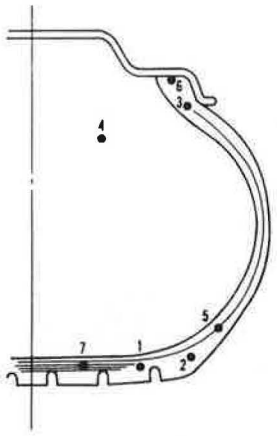
1. Tread wear measurements after 8,833 mi over the test course, which included operation over the severe rock section.
2. Durability determinations of all 36 tires involved in the test. This included both visual observations and X-ray examination of the tire carcass.
3. Tire thermal profile measurements. These were performed using six tires outfitted with thermocouples (Figure 3).
4. Tire bead unseating determinations. Because of the lower pressures, a concern was raised about the tires losing contact with the rims. Visual observations were made during the test to see if this was a problem.

Results

Tread wear results are shown in Table 1. These results are expressed in miles per 32nd of an inch of rubber (mi/32).

The radial tires operated over the closed-loop circuit at reduced pressures experienced approximately 15 percent less tire tread wear than the tires operated at the standard highway pressure.

Tire tread durability results are shown in Table 2. All 36 radial tires applied to the two log trucks at the beginning of the test completed the 8,900 scheduled miles without failure



Thermocouples

Location	Description
No.1	Belt Edge
No.2	Shoulder
No.3	Bead
No.4	Air Cavity
No.5	Carcass Ply
No.6	Bead Bundles
No.7	Belt Plies

FIGURE 3 Locations of thermocouples for NATC project.

or air loss. On the closed-loop course, the high-pressure tires experienced approximately three times as many tread rib cuts, including five cut penetrations to the protector ply, as compared to the lower-pressure tires, which experienced no penetrations to the protector ply. Several of the high-pressure tires would have failed from the number of rocks jammed between the duals if the rocks had not been removed quickly. No rock jamming was experienced by the low-pressure duals.

Tire bead temperature results are depicted in Figure 4. Excessive tire temperatures were not a factor influencing tire durability in this controlled program because average test speeds were below 30 mph when the tires were deflated and thermal profiles showed no excessive heat build up at 20 percent deflection at speeds up to 40 mph.

Bead stability for the higher tire deflection (20 to 22 percent) at speeds used in this test were acceptable. No bead seating problems were encountered at 25 psi.

Tire Manufacturer Testing

Both the Goodyear and Michelin Companies provided tires for this test. Both corporation's representatives examined the test tires on completion of the test. The examinations by both firms noted no significant differences between tires inflated at the standard highway pressure and those operated at reduced pressure. In a letter forwarded to the Forest Service, Michelin reported the following:

> The information obtained from the NATC proof of concept test does not indicate a loss of performance in the areas of tread aggression, wear, or crown injury as a result of running at the specified reduced pressures and speeds. The tests, having been of short duration, don't provide an indication of long-term carcass endurance. We do feel that the reduced pressure will have an ultimate effect on bead cracking, radial splits, etc. This tendency will be reduced in the logging application where carcasses are normally retired from service for other reasons.

Goodyear stated, "Examination of the 18 tires revealed no abnormalities after completion of the 8,900-mi NATC test."

Goodyear had similar concerns regarding long-term effects of reduced pressure operation and decided to initiate a rigorous field evaluation.

GOODYEAR FIELD EVALUATION TEST

Background

The Goodyear Tire and Rubber Company initiated an extensive tire evaluation of standard highway pressure tires (99 psi

TABLE 1 COMPARISON OF TREAD WEAR BETWEEN STANDARD- AND REDUCED-PRESSURE TIRES OPERATING OVER THE NATC TEST COURSE

	Standard Highway Pressure Vehicle (8-12% Deflection)		*Reduced Pressure Vehicle* (20-22% Deflection)	
Tire Position	Avg. % Worn	Miles/32	Avg. % Worn	Miles/32
#1 Axle, Steer Tread Wear, 2 Tires	44.0	1081	43.8	1085
#2 Axle, Drive Tread Wear, 4 Tires	27.9	1736	24.1	2016
#3 Axle, Drive Tread Wear, 4 Tires	54.2	883	45.9	1052
#4 Axle, Trailer Tread Wear, 4 Tires	17.1	2367	14.6	2815
#5 Axle, Trailer Tread Wear, 4 Tires	31.8	1291	28.6	1440

TABLE 2 COMPARISON OF TIRE DAMAGE BETWEEN TEST VEHICLES OPERATED AT NATC IN TERMS OF NUMBERS OF INJURIES ON COMPLETION OF TEST

	Standard Highway Pressure Vehicle	*Reduced Pressure Vehicle*
Tire Damage Noted	8-12% Deflection	20-22% Deflection
Undercut Flex Cracks	504	303
Rib Cuts	147	49
Rib Cuts to Belt	5	0
Tread Cut Chunkouts	171	35

hot) versus tires run on CTI equipped logging trucks during January 1989. Two fleets, FJM Trucking of Roseburg, Oregon, and Foglio Trucking of Florence, Oregon, were chosen as the test fleets for control versus CTI tires.

Tread depth data were gathered using an electronic data collection system. Each tire involved in this field evaluation was uniquely numbered and its position known at all times.

There were some concerns raised that the standard highway pressure tires would benefit from the pneumatic rolling effect of the reduced pressure tires. In other words, the highway pressure tires would realize some of the benefits of the CTI tires by having a smoother road to traverse. It was decided to note this fact, but to proceed with the test anyway since there was no feasible way to gather actual field data without pairing the CTI and non-CTI vehicles to operate over the same road.

FJM Trucking

At FJM Trucking of Roseburg, Oregon, the test tires ran on Kenworth log trucks, which had 400-hp engines. Two separate tests were run at FJM. The first test ran from January through April 1989. One CTI truck ran directly against one truck without CTI (the control vehicle). The two trucks ran side by side, always going to the same timber landings.

A typical trip for FJM would involve spending up to 2 hr on an Interstate highway at 55 mph, then 20 to 40 min off the highway at speeds under 35 mph to the landing site. On the average, three round trips per day were made by FJM. Less than 10 percent of the tire life was spent off the paved highway.

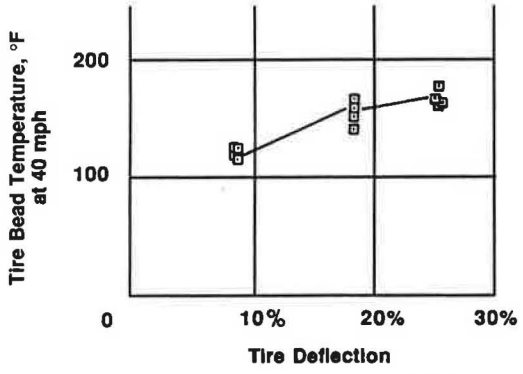


FIGURE 4 Bead temperatures for 11R24.5 drive tires.

In the first test at FJM, the control tires were set at 99 psi hot for all wheel positions (steer, drive, trailer). The CTI tire pressure varied depending on load and speed. When the vehicle was operating on the highway at speeds over 35 mph and fully loaded, the tire pressures were set to the same level as the control vehicle. When the CTI truck ran at speeds under 35 mph and fully loaded, the steer tires were adjusted to a reduced pressure of 90 psi, and the drive and trailer tires were adjusted to a reduced pressure of 57 psi.

When the CTI truck ran at speeds under 35 mph with an empty load, the steer tires were adjusted to a reduced pressure of 90 psi (same as fully loaded), and the drive tires were adjusted to a pressure of 28 psi. Trailer tire pressure was not a factor when the vehicle contained an empty load because trailers ride to the landing "piggyback."

Figure 5 shows the steer tire test results when the CTI tires were reduced to a pressure of 90 psi off-highway. The control tires averaged 26,060 mi versus 23,190 mi for the CTI tires. Miles/32nd of an inch was 1,446 versus 1,262, respectively. Statistically, at a 95 percent confidence level, there was no difference in treadwear.

Figure 6 shows the results of the second test at FJM, which evaluated only steer tires. Tire pressure was reduced down to 70 psi when off the highway and at speeds under 35 mph. Tires ran from April through August 1989. Treadwear increased approximately 15 percent from the first test for both control

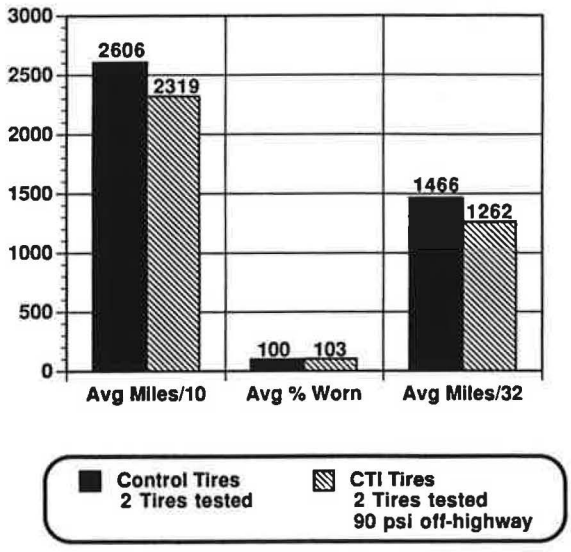


FIGURE 5 Load per tire on steer tires with CTI tires at 90 psi, off-highway (FJM Trucking Co.).

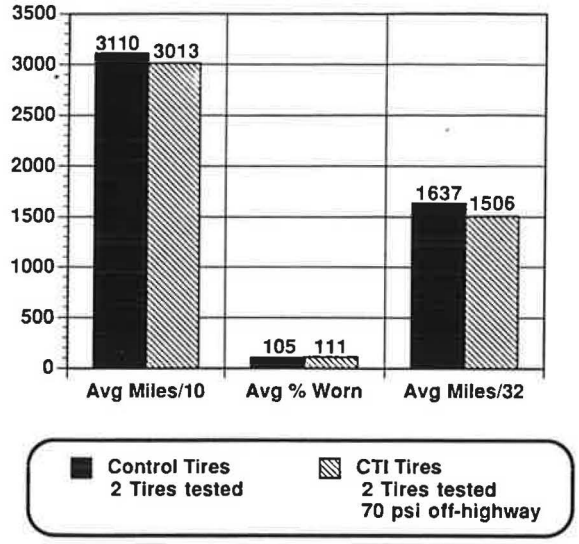


FIGURE 6 Load per tire on steer tires with CTI tires at 70 psi, off-highway (FJM Trucking Co.).

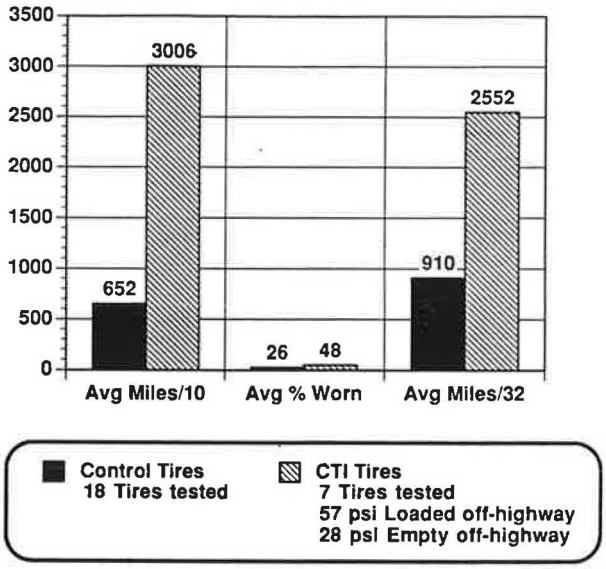


FIGURE 8 Load per tire on drive tires for Retread 1 with CTI tires at 57 psi, loaded, off-highway, and 28 psi, empty, off-highway (FJM Trucking Co.).

and CTI tires. This was primarily because of two factors, a change in landing site and a seasonal effect. The control tires averaged 1,637 versus 1506 mi/32nd of an inch for the CTI tires. Once again, at a 95 percent confidence level, there was no difference in treadwear at this reduced pressure.

Figure 7 shows the drive tire data. Control tires averaged 70,930 mi to removal versus 67,980 mi for the CTI tires. Tread wear was almost identical at 2,622 versus 2,641 mi/32nd of an inch, respectively.

After tires have finished running through their original life, tires are holographed, inspected, and then retreaded. Steer and drive tires are both capped with a G188 drive tire design and run on the drive position. Figure 8 shows the latest data of Retread 1 on the drive position. It is too early to draw any conclusions as to treadwear rating because mileages are still early and not all tires were inspected. Therefore, all tires were retreadable, and holographic analysis revealed equivalent residual durability after running through its original tire life.

Figure 9 shows the trailer tire data. These tires are just beginning to come out of service before getting their first retreading. Tread wear was 2,319 mi/32nd of an inch for the control vehicle versus 2,920 mi/32nd of an inch for the CTI vehicle.

Foglio Trucking

At Foglio Trucking of Florence, Oregon, two control trucks were run versus two trucks with CTI. A typical trip for a

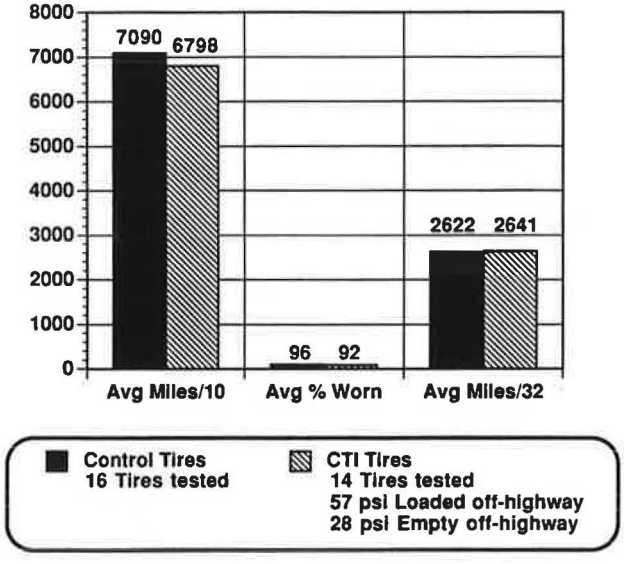


FIGURE 7 Load per tire on drive tires with CTI tires at 57 psi, loaded, off-highway, and 28 psi, empty, off-highway (FJM Trucking Co.).

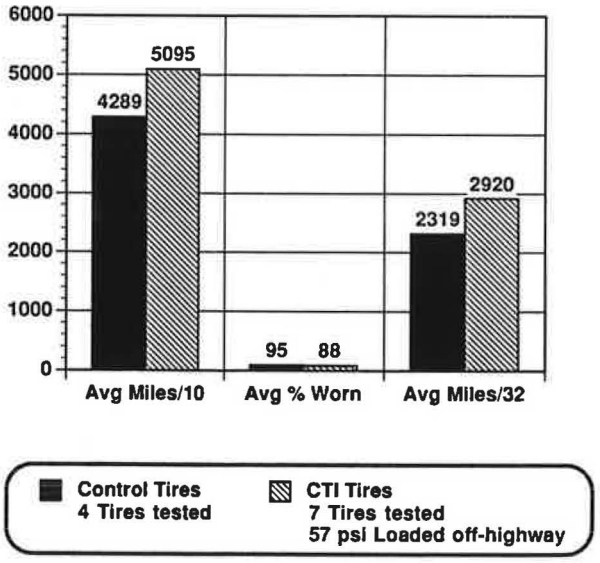


FIGURE 9 Load per tire on trailer tires with CTI tires at 57 psi, loaded, off-highway (FJM Trucking Co.).

Foglio truck would involve 20 to 30 min on the paved highway at 55 mph followed by 20 to 40 min off the highway at speeds under 35 mph. Over 35 percent of the tire life is spent off the highway. Control trucks operated on the same roads as the CTI trucks.

Steer, drive, and trail tires on the control trucks were set at 99 psi hot. CTI trucks, when on the highway at speeds over 35 mph, also operated at 99 psi hot. When the CTI trucks were off the highway at speeds under 35 mph, steer tires were set at 82 psi, drive tires at 57 psi hot when loaded, and 28 psi hot when empty. Trailer tires on the CTI trucks were set at 57 psi hot when off the highway and under 35 mph.

Tires were mounted January 1988 and ran during most of the year. Figure 10 shows that the steer tires averaged 34,670 mi to removal on the control trucks versus 33,470 mi on the CTI vehicles. Tread wear in mi/32nd of an inch was 1,860 on the control versus 1,700 for CTI. Statistically, at 95 percent confidence level, there was no significant difference in treadwear.

These tires were retreaded and are now running as drive tires (G188 design). Data are very early and are shown in the next graph.

Figure 11 shows drive tire data. The control tires averaged 1,955 mi/32nd of an inch versus 1,725 mi/32nd of an inch for CTI tires. Once again, statistically at 95 percent confidence level, there was no difference in treadwear.

Figure 12 shows the first retreading of the drive tires. The procedure for inspection and holographing was identical to the FJM operation. All tires were retreadable with no major defects noted.

Trailer data are shown in Figure 13. At 50 percent worn, control tires are averaging 4,316 mi/32nd of an inch versus 4,431 mi/32nd of an inch for tires run on CTI vehicles. At 95 percent confidence level, there was no difference in treadwear.

Retreadability

Tire retreadability is an important factor in the CTI equation. In logging service, tires generally are recapped three or four times. As part of the Goodyear field evaluation, all tires that are being followed will be retreaded as many times as possible. It is important to determine what, if any, effect CTI may have on running tires with reduced pressure and increased deflection.

All tires at both Foglio Trucking and FJM Trucking were retreadable after running their original life. Holograph data revealed no difference so far between control and CTI tires. It will take up to 3 years before multiple retreadability studies will be completed.

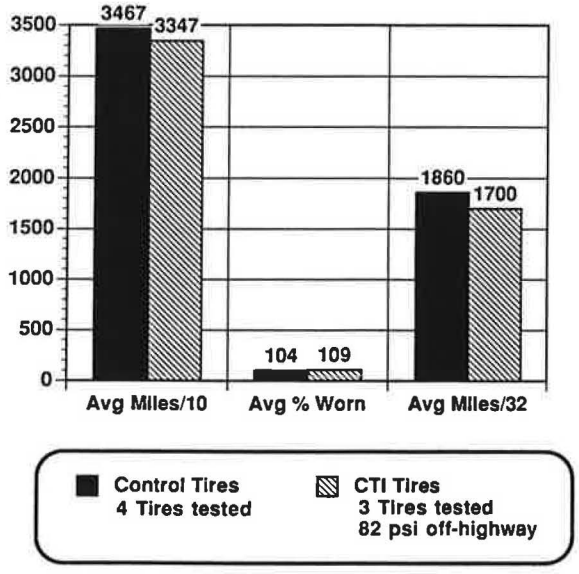


FIGURE 10 Load per tire on steer tires with CTI tires at 82 psi, off-highway (Foglio Trucking Co.).

Field Evaluation Conclusions

The Goodyear Tire and Rubber Company has the following conclusions from its field evaluation:

- Original tread water was equivalent for the tires which operated with or without CTI;
- At the end of original tire life, CTI tires were acceptable for retreading;
- Tread wear varies significantly from location to location, i.e., low, moderate, and fast tire wear rates; and
- Because of many uncontrolled test variables, only tires run at the same time, at the same location, using the same roads, can be directly compared.

SUMMARY AND CONCLUSIONS

Currently, it appears that there are no detrimental effects of reduced tire pressure operation on tread wear provided speeds are reduced accordingly. The NATC study indicated a decrease in tire wear for tires operating under reduced tire pressure, whereas Goodyear's study indicated that it was too close to make any conclusive statement one way or the other. The severity of the NATC test course, which was constructed to

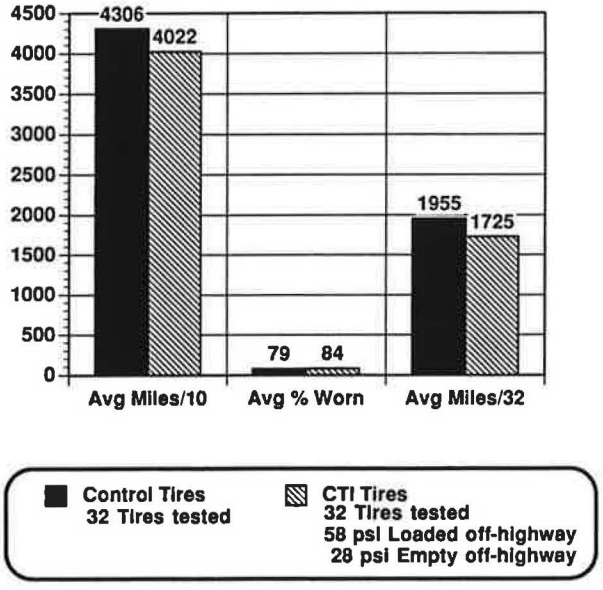


FIGURE 11 Load per tire on drive tires with CTI tires at 58 psi, loaded, off-highway, and 28 psi, empty, off-highway (Foglio Trucking Co.).

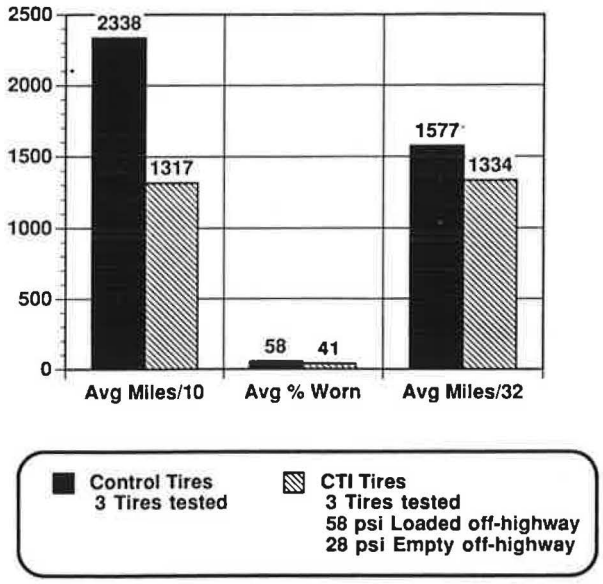


FIGURE 12 Load per tire on drive tires for Retread 1 with CTI tires at 58 psi, loaded, off-highway, and 28 psi, empty, off-highway (Foglio Trucking Co.).

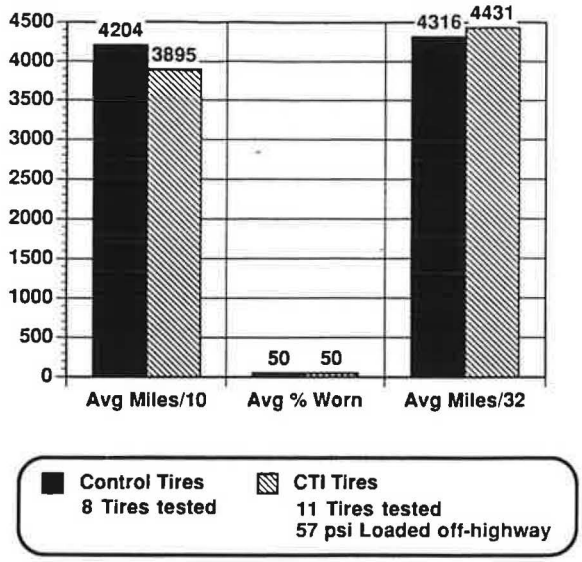


FIGURE 13 Load per tire for trailer tires with CTI tires at 57 psi, loaded, off-highway (Foglio Trucking Co.).

simulate accelerated wear on vehicles and tires, contrasted with less severe field conditions. This difference may account for the variation in results.

The actual amount of tread wear one might obtain is dependent on many factors. Certainly the abrasive quality and particle size of rock aggregate used as surfacing material heavily influence tire wear. Operation at reduced tire pressure would probably exhibit slightly less tread wear over severely washboarded aggregate roads, or roads with numerous potholes. This is because of the softer ride of the reduced-pressure tires, which results in less energy being used to damage the road and tires. Conversely, a smooth surface might cause an increase in tread wear if the tire pressure was reduced beyond the optimum point for efficient traction. The goal of a properly installed CTI system is to match the correct tire pressure to the vehicle's load, speed, and road condition. In any case, it appears that there are no significant detrimental effects on tread wear from operating under reduced tire pressure.

Conclusive results for reduced-pressure tire carcass life have not been determined. While current tests indicate no detrimental effects, it is too early to make predictions as to the outcome.

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2. H. C. Hodges et al. *Nevada Automotive Test Center Final Report—Central Tire Inflation*. Report 53-9JA9-6-SD647. U.S. Department of Agriculture, Forest Service, Technology and Development Center, San Dimas, Calif., 1987.