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Transportation Research Record 1291

Contents—Volume 1

Sub-Saharan Africa

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Transportation Research Record 1291

Contents—Volume 2

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 (−).

a. Terrain Data

	1	2	3	4	5	6
6	652	612	636	661	679	655 D
5	645	588	594	612	648	606
4	618	576	591	600	603	594
3	600	570	597	600	573	606
2	594	570	582	594	564	600
1	605 O	606	564	597	564	600

b. Cut-Fill Data (5%)

	1	2	3	4	5	6
6	0.0	+6.9	-10.4	-5.5	-15.2	0.0 D
5	-5.0	+15.6	+9.3	+16.4	-5.0	+37.9
4	0.0	+7.5	-6.6	+9.2	+10.8	+26.5
3	-3.0	+0.8	-21.9	-8.3	+21.5	-4.8
2	+0.3	+7.7	-7.6	0.0	+33.4	0.0
1	0.0 O	-23.5	+17.2	0.0	+33.6	0.0

c. Construction Surface (5%)

	1	2	3	4	5	6
6	652.0	618.9	625.6	655.5	663.8	655.0 D
5	640.0	603.6	603.3	628.4	643.0	643.9
4	618.0	583.5	584.4	609.2	613.8	601.2
3	597.0	570.8	575.1	591.7	594.5	601.2
2	594.3	577.7	574.4	594.0	597.4	600.0
1	605.0 O	582.5	581.2	597.0	597.6	600.0

Legend: O - Grid Point; O - Origin; D - Destination

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.

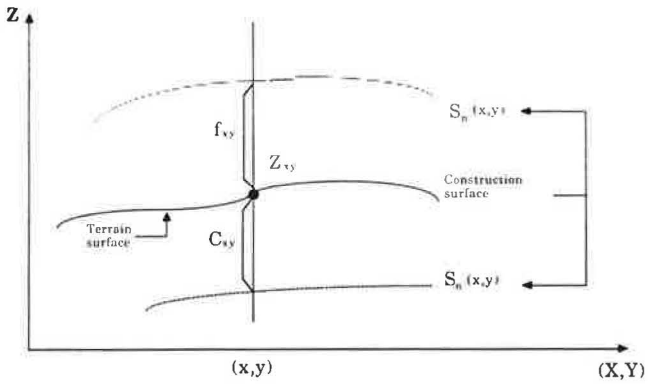
The link-evaluation submodel uses the BCM parameters along with other socioeconomic data. For example, construction quantities (to formation level) are estimated from the cuts and fills at the grid points and multiplied by unit costs to arrive at estimates of earthwork costs; from the construction surface 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:

$$\frac{\partial S_n(i,j)}{\partial x} \le g \quad \forall\, i,j$$

$$\frac{\partial S_n(i,j)}{\partial x} \le g \quad \forall\, i,j$$

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 : $S_n(x,y) = a_0(0) + a_1x + a_2y\,(1) + a_3x^2 + a_4xy^2$

$$+ a_5y^2(2) + a_6x^2 + a_7x^2y + a_8xy^2$$

$$+ a_9y^3(3) + \cdots\cdots$$

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

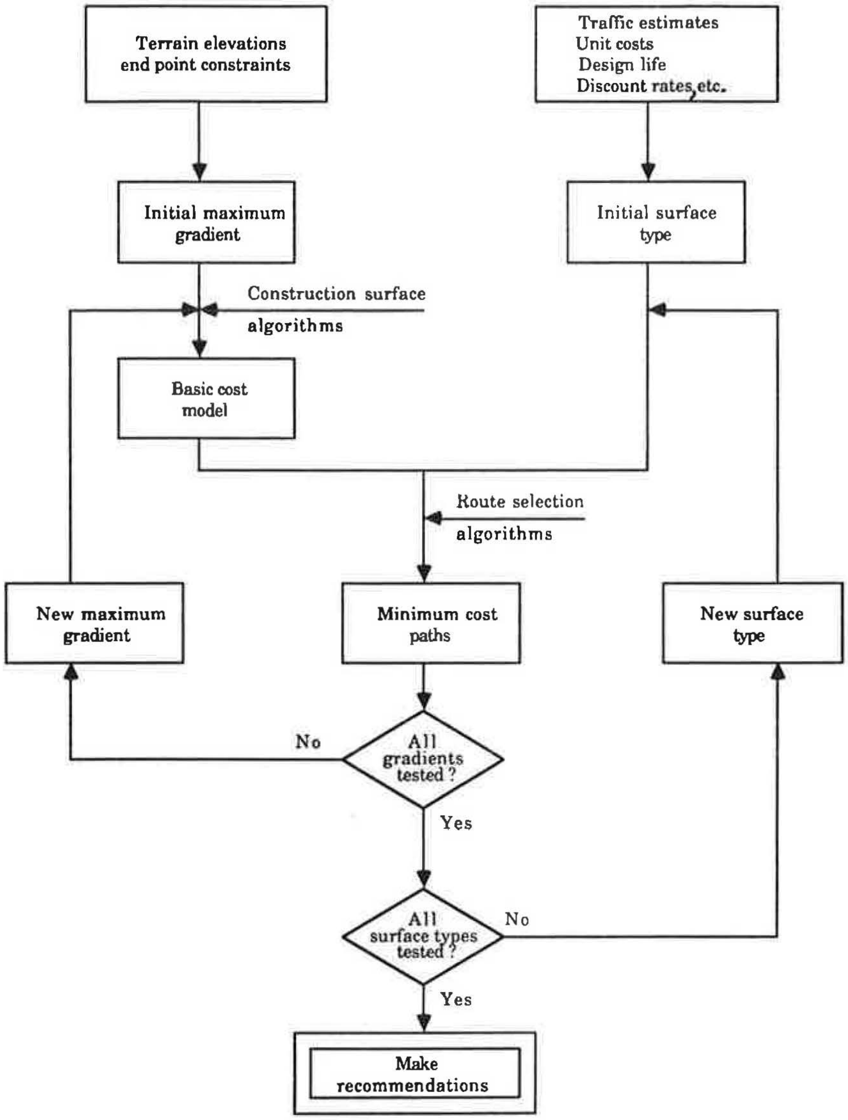


FIGURE 3 Procedure for determination of line and grade.

```
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

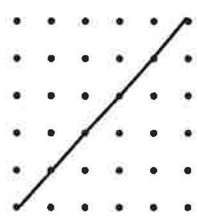


FIGURE 5 Facsimile output for vehicle operating costs: Path No. 1.

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

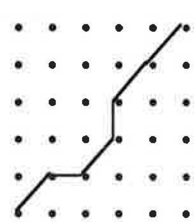


FIGURE 6 Facsimile output for vehicle operating costs: Path No. 2.

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

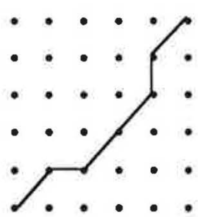


FIGURE 7 Facsimile output for vehicle operating costs: Path No. 3.

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

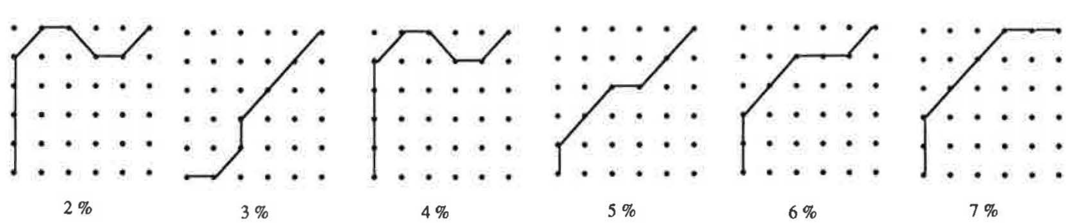


FIGURE 8 First-minimum construction cost routes for sample problem.

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

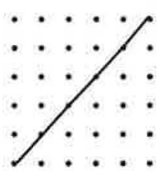


FIGURE 9 First-minimum vehicle operating cost routes for sample problem.

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

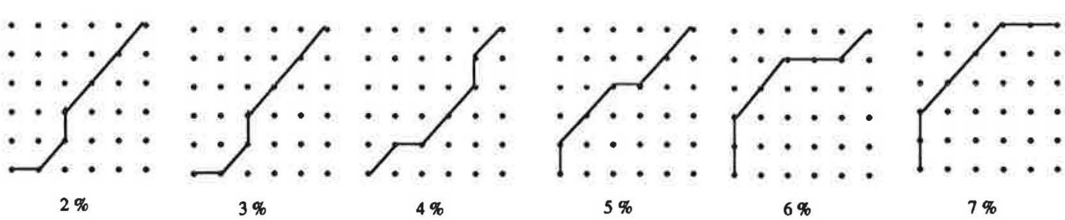


FIGURE 10 First-minimum total-cost routes for sample problem.

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

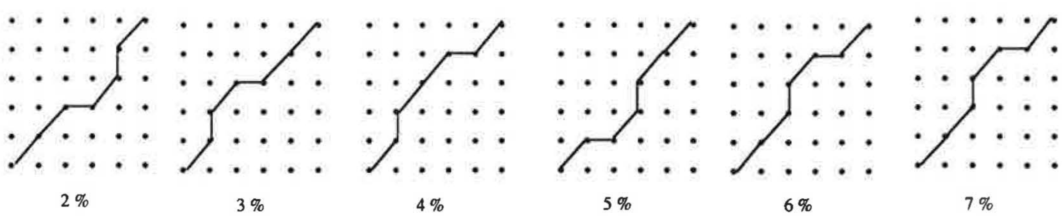


FIGURE 11 Second-minimum vehicle operating cost routes for sample problem.

(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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2. F. J. Gichaga and N. A. Parker. *Essentials of Highway Engineering*. Macmillan, London, 1988.

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.

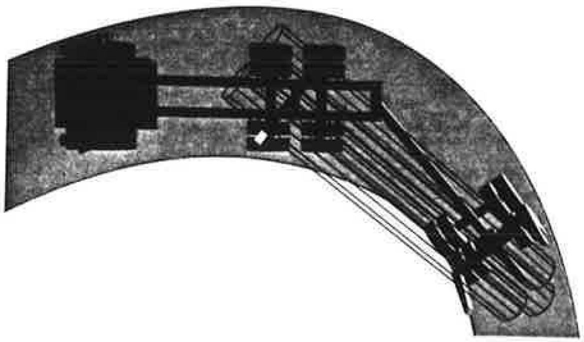

FIGURE 1 Perspective view looking down on a loaded log truck-tractor with pole-steered trailer climbing around a steep curve.

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.

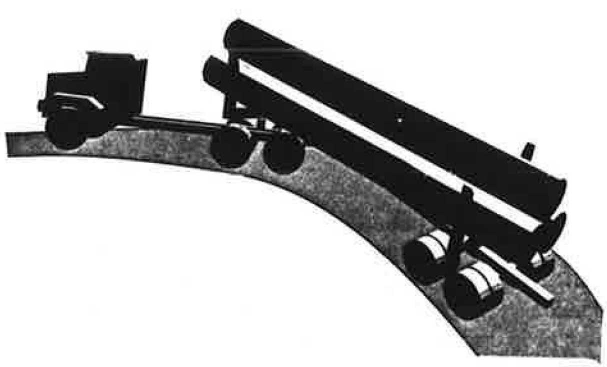

FIGURE 2 Inside-of-curve perspective view of a loaded log truck climbing around a steep curve.

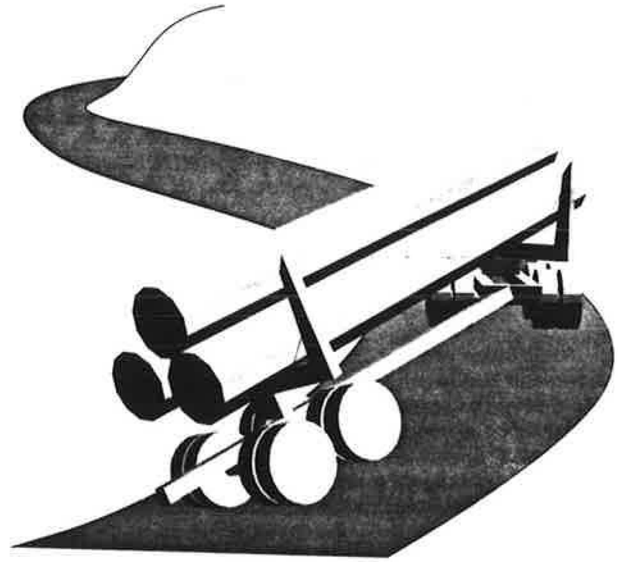

FIGURE 3 Outside-of-curve perspective view of a loaded log truck climbing around a steep curve.

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

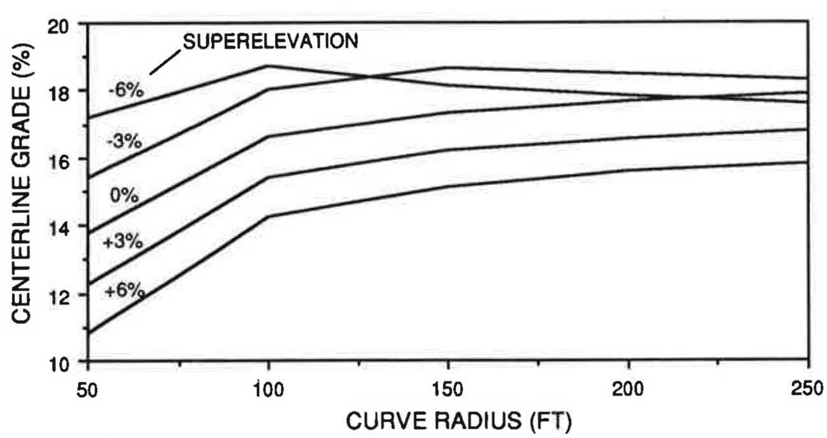

FIGURE 4 Gradeability as a function of curve radius and superelevation for a coefficient of traction of 0.45.

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

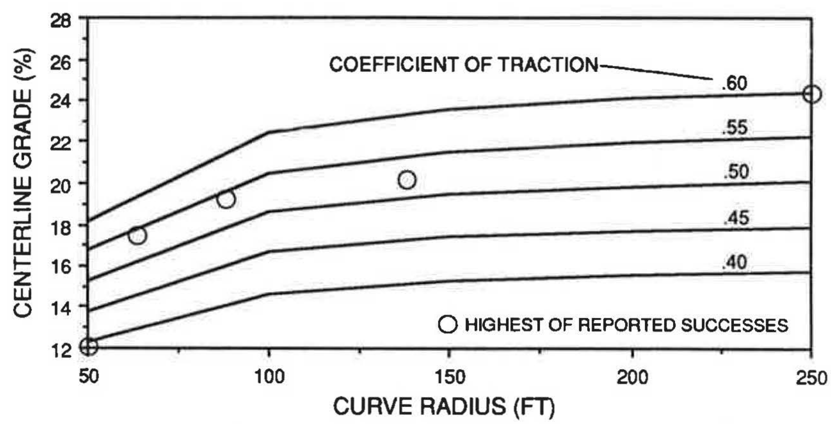

FIGURE 5 Gradeability as a function of traction coefficient, curve radius, and zero superelevation. Points of highest reported successful operations are identified.

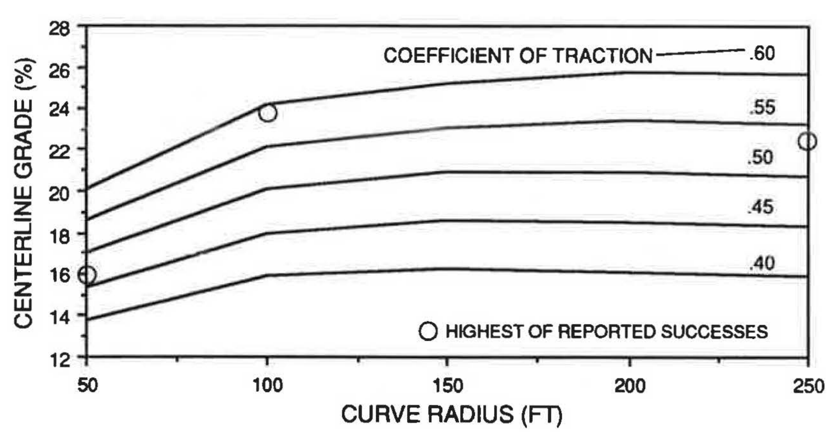

FIGURE 6 Gradeability as a function of traction coefficient, curve radius, and −3 percent superelevation. Points of highest reported successful operations are identified.

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 average 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 arguments (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.

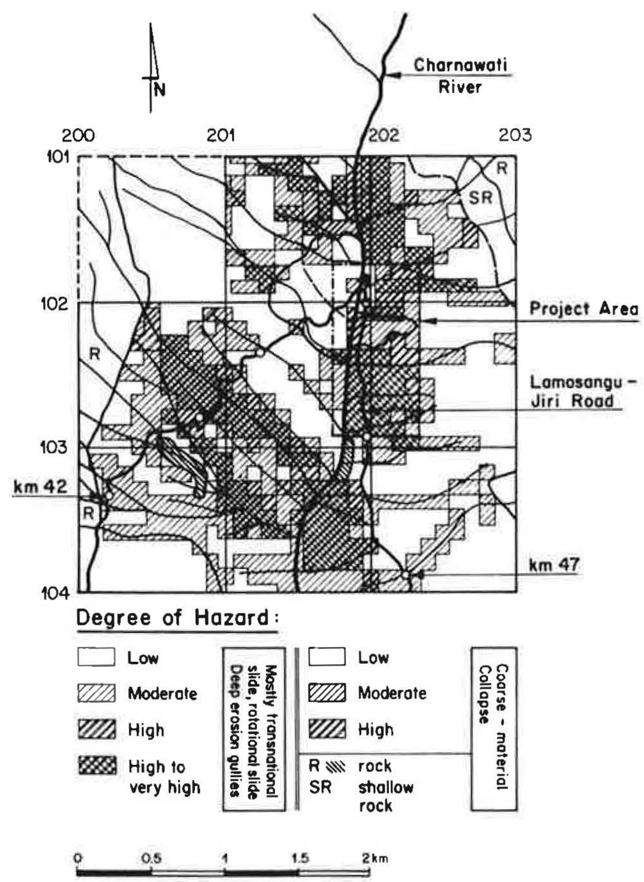

FIGURE 3 Charnawati Valley soil hazard map.

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

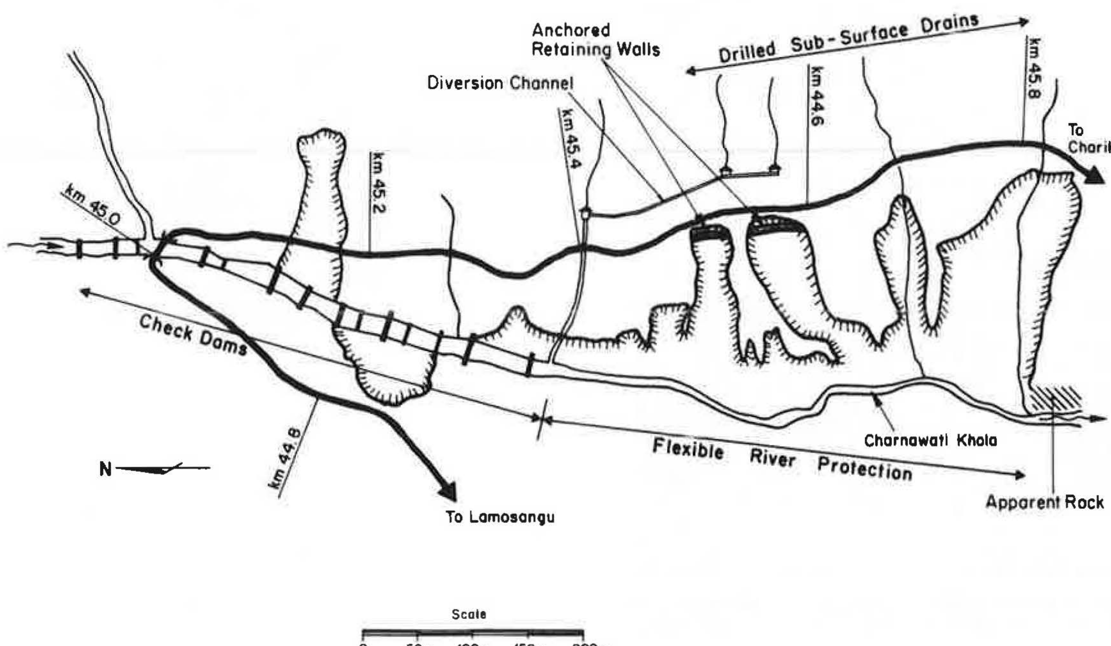

FIGURE 4 Charnawati Project: overall plan view. Only the main substantial structures are presented; drainage and biotechnical measures in the slides are not shown.

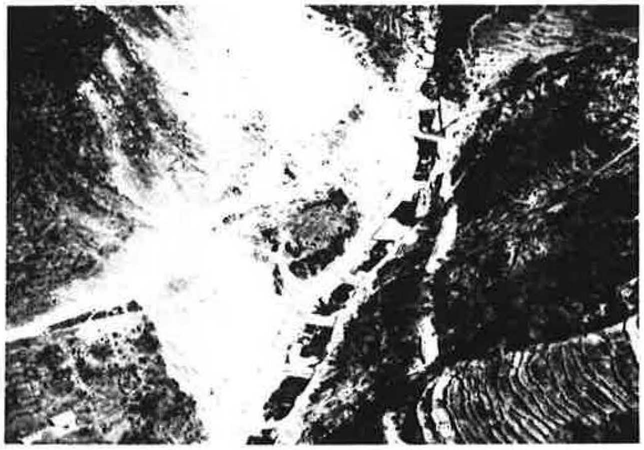

FIGURE 5 Aerial photograph of check dams and road crossing. Note the slide on the right-hand bank where the alignment had to be shifted to the mountainside. The cleared area to the left of the bridge was used for quarrying.

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 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.

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.

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

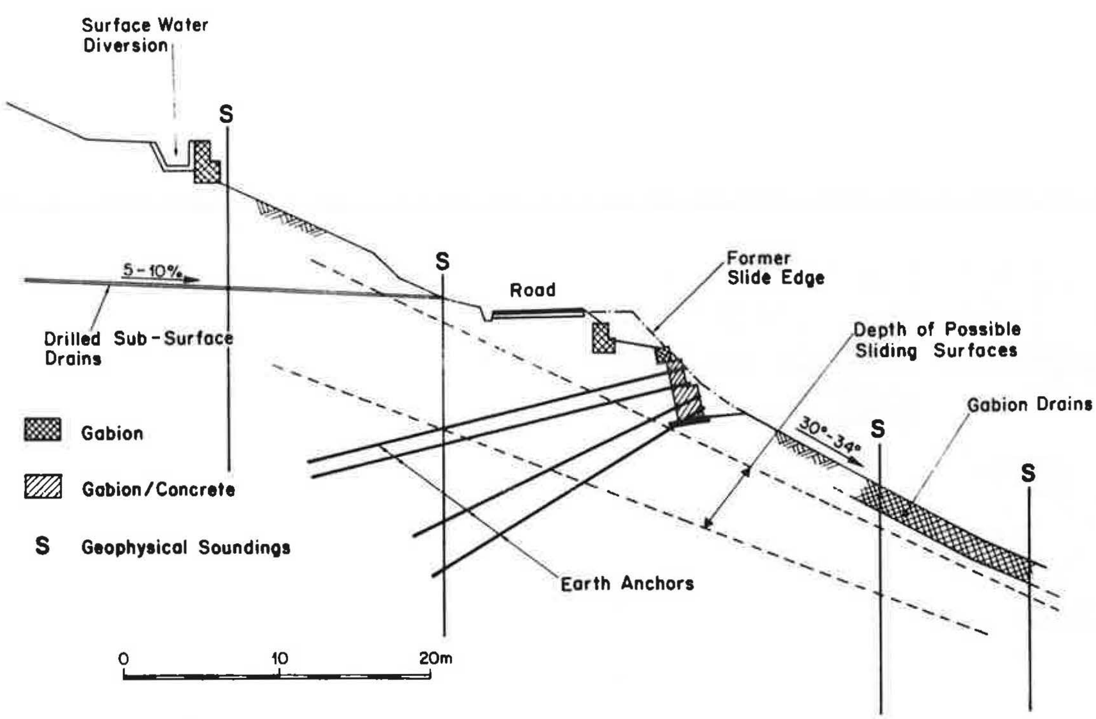

FIGURE 9 Cross section of slide stabilization measures at Km 45.4.

- 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.

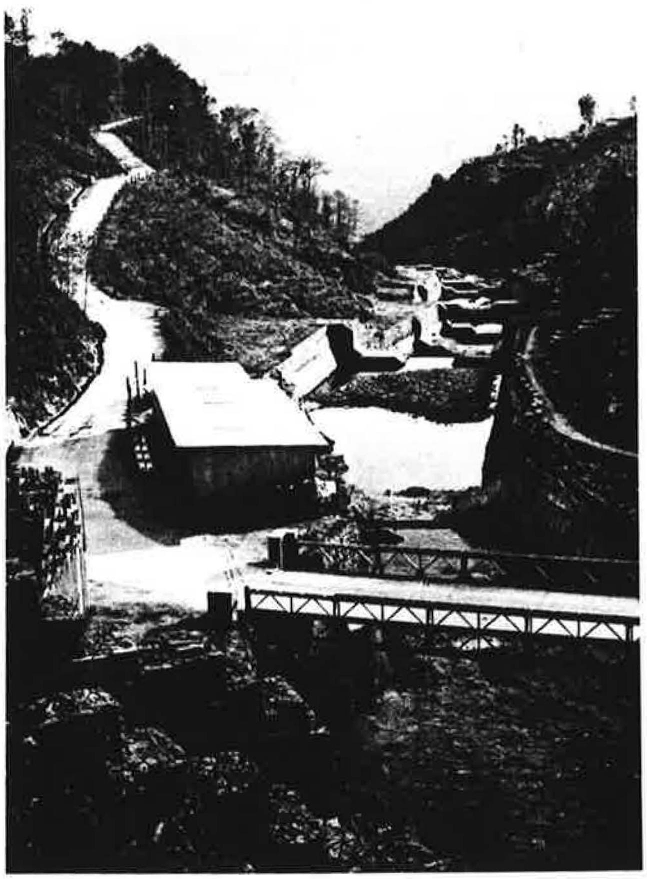

FIGURE 10 New Charnawati bridge crossing. The store in the foreground is used for current construction work.

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

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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.

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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;

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.

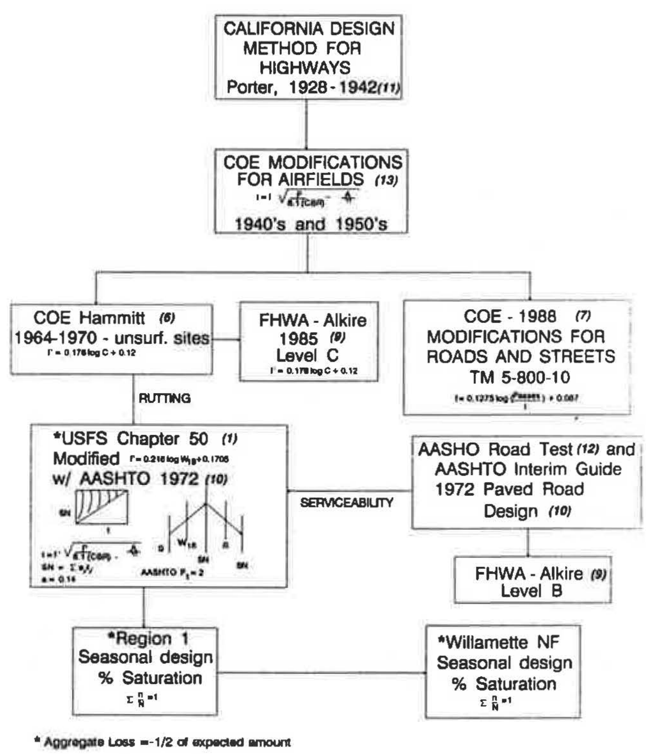

FIGURE 1 Design method relationships originating from California for aggregate-surfaced and earth roads.

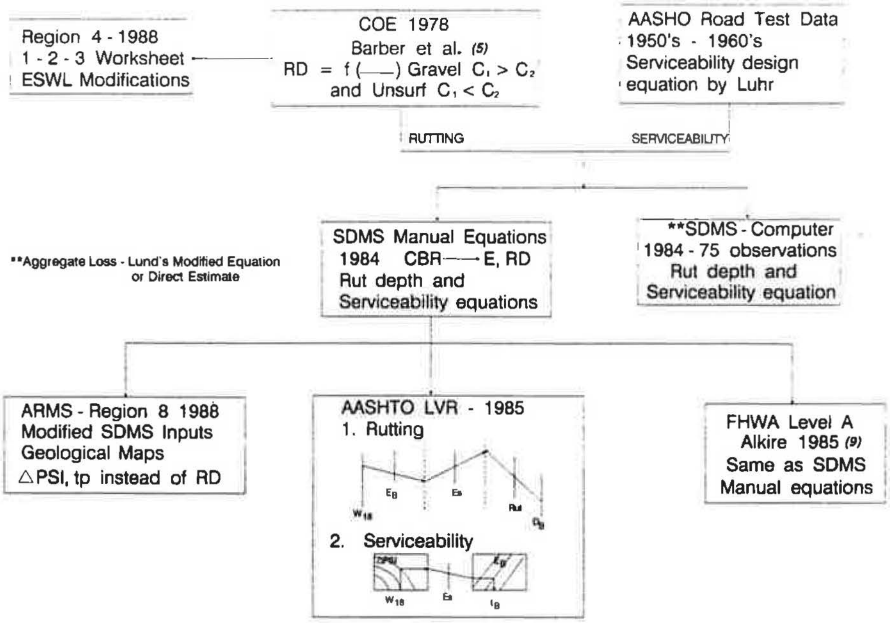

FIGURE 2 Design method relationships originating from the Corps of Engineers for aggregate-surfaced and earth roads.

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

$$t = f\left[\frac{P}{8.1\,(\mathrm{CBR})} - \frac{A}{\pi}\right]^{0.5} \qquad (1)$$

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:

$$f' = t\left[\frac{P}{8.1\,(\mathrm{CBR})} - \frac{A}{\pi}\right]^{-0.5} \qquad (2)$$

It was then possible to plot f' against the failure coverage and, through linear regression analysis, obtain the following relationship:

$$f' = 0.176 \log C + 0.12 \qquad (3)$$

where the terms are as previously defined. Substituting f' for f in the original thickness equation (1) results in the following:

$$t = (0.176 \log C + 0.12)\left[\frac{P}{8.1\,(\mathrm{CBR})} - \frac{A}{\pi}\right]^{0.5} \qquad (4)$$

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

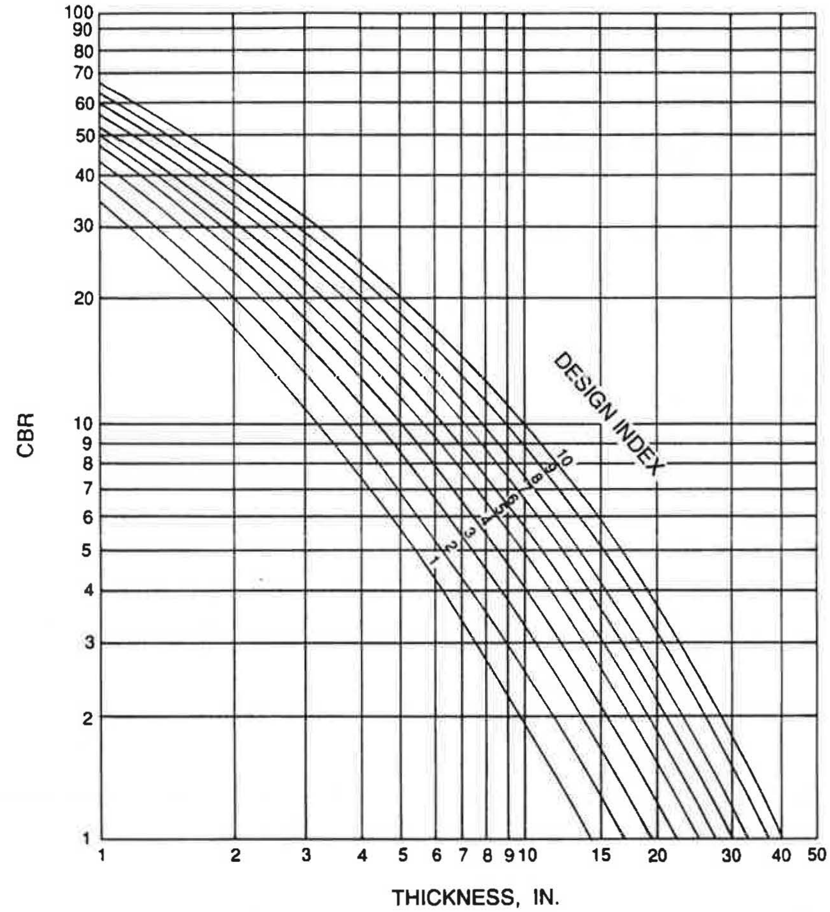

FIGURE 3 Design curves for gravel-surfaced roads (*7*).

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:

$$\text{ESWL} = C1 * \text{Lg} + C2 * \text{Lg} * \text{Da} \tag{7}$$

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

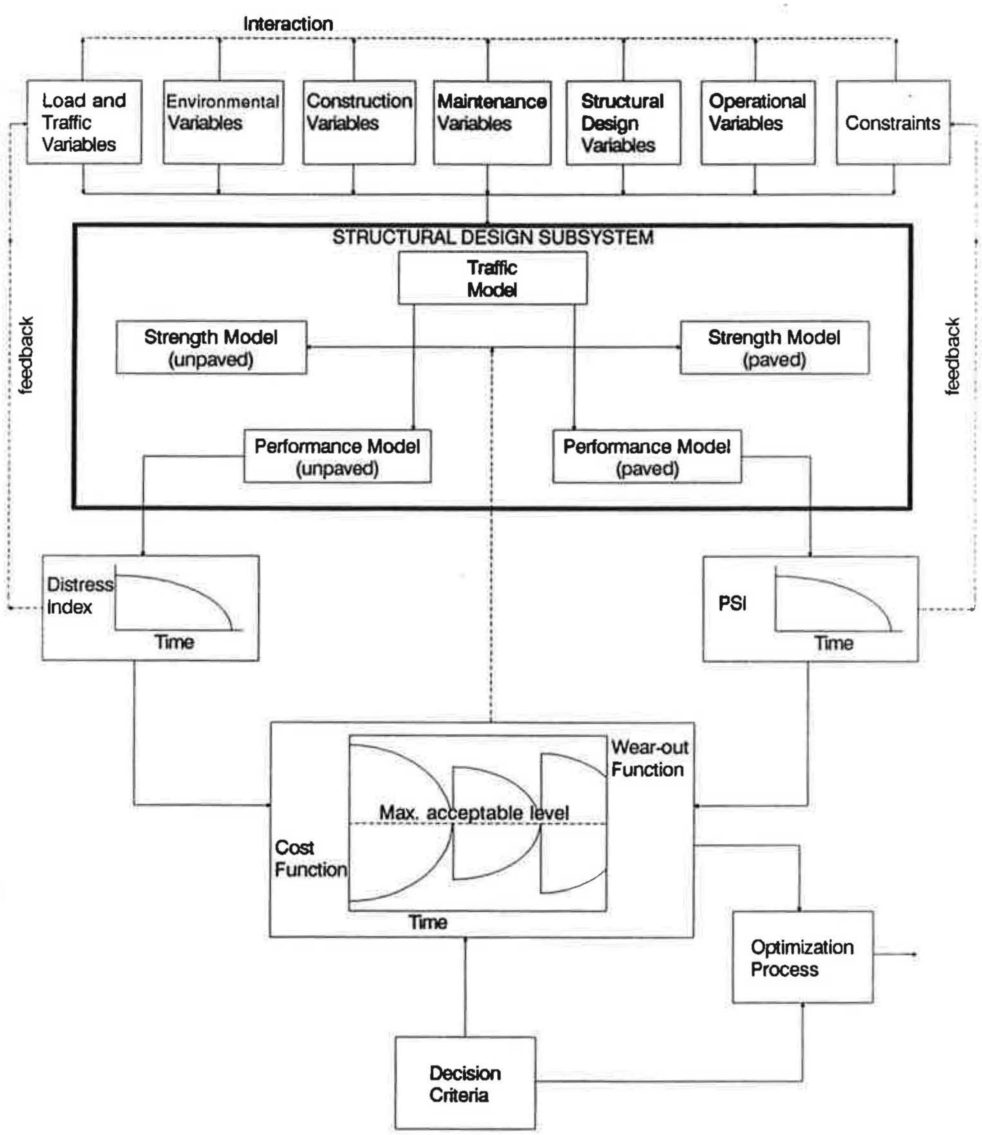

FIGURE 4 Conceptual system for low-cost roads (*16*).

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

$$W_{18R} = 0.1044 * \mathrm{RUT}^{2.575} * \log_{10} \mathrm{THICK}^{5.155} * \left(\frac{E_1}{1{,}800}\right)^{3.434} * \left(\frac{E_2}{1{,}800}\right)^{1.048} \quad (9)$$

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

$$W_{18\mathrm{RUT}} = 64.51 * (t_p)^{-1.4665} * [3\,(\mathrm{TSI})^{-0.5}]^{2.575} * \log t^{5.155} * \left(\frac{E_B}{1{,}800}\right)^{3.434} * \left(\frac{E_{SG}}{1{,}800}\right)^{1.048} \quad (10)$$

where

$W_{18\mathrm{RUT}}$ = 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.

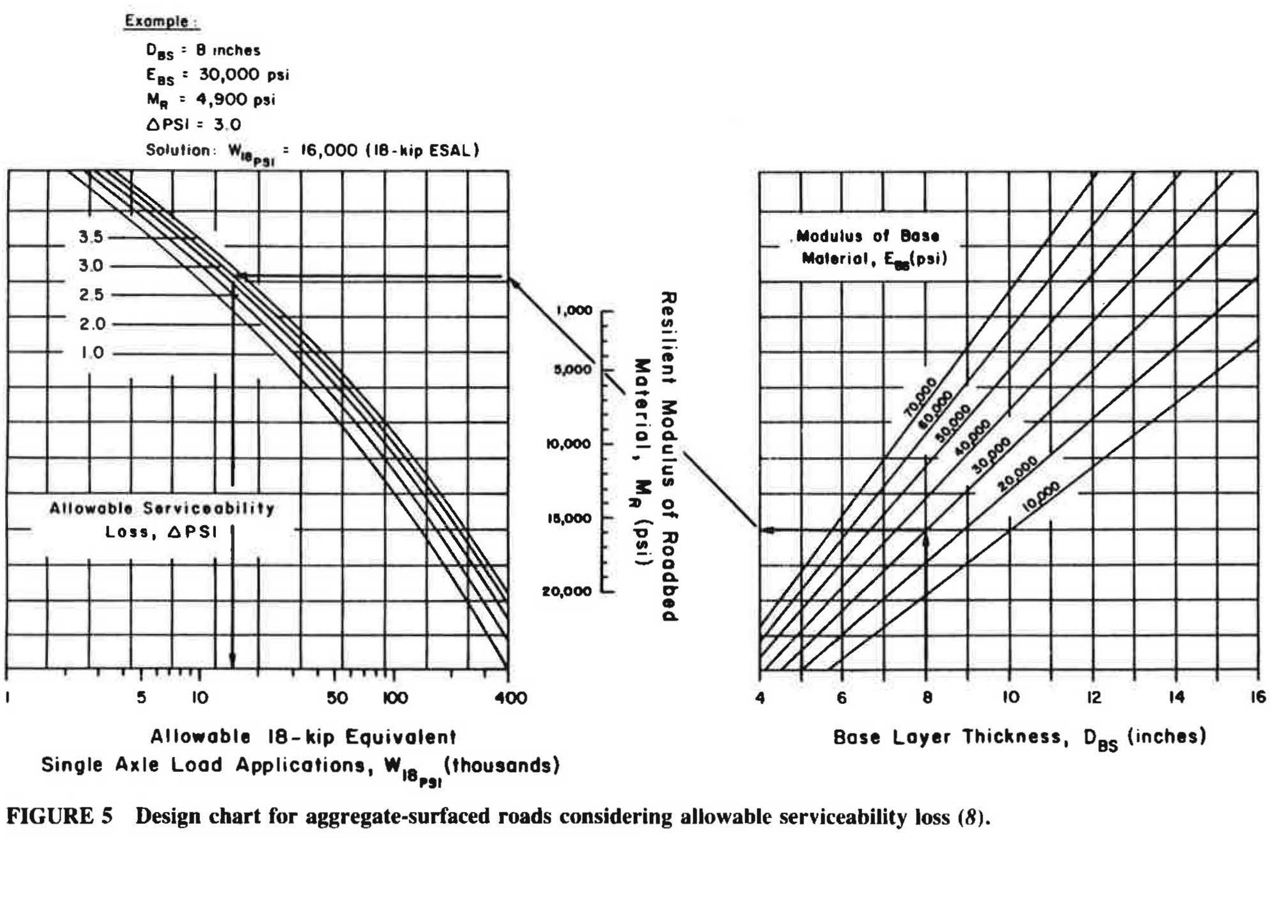

FIGURE 5 Design chart for aggregate-surfaced roads considering allowable serviceability loss (*8*).

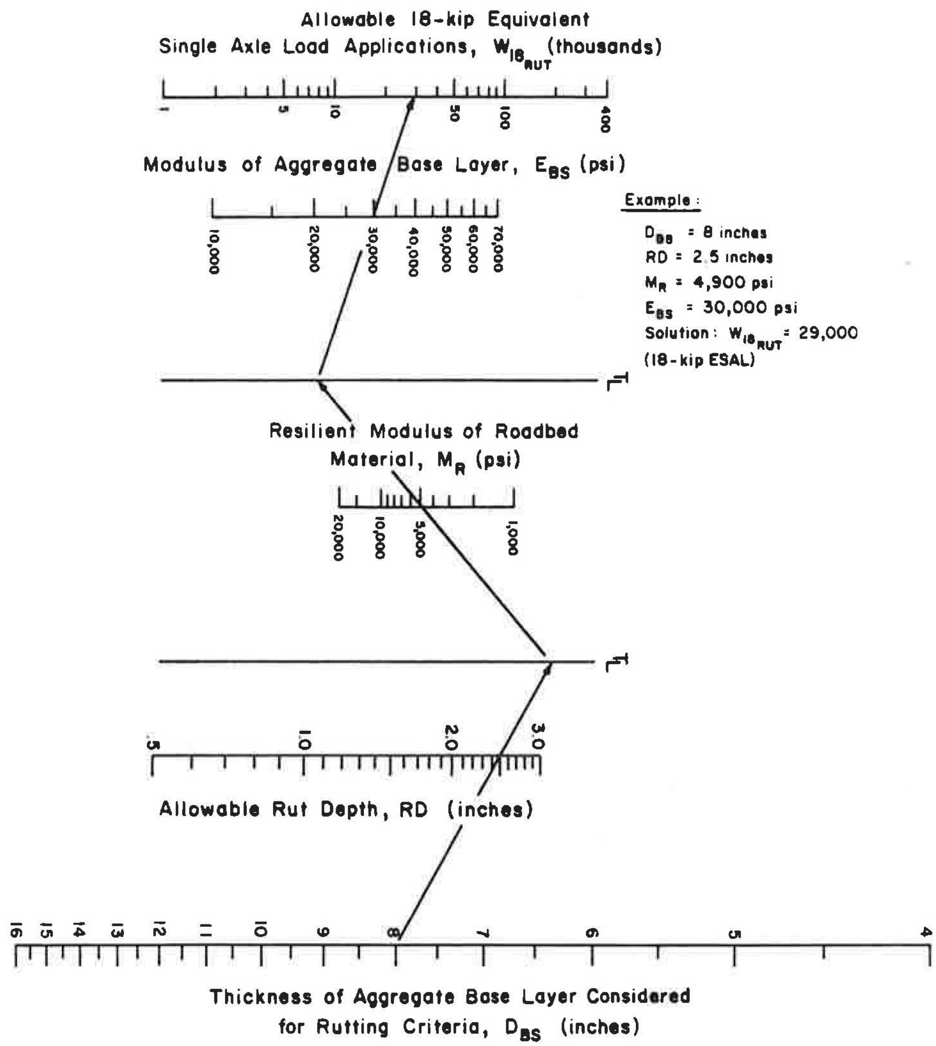

FIGURE 6 Design chart for aggregate-surfaced roads considering allowable rutting (*8*).

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:

$$\log W_{t18} = 9.36 \log(\mathrm{SN} + 1) - 0.20 + \frac{G_t}{0.40 + \dfrac{1{,}094}{(\mathrm{SN} + 1)^{5.19}}} + \log \frac{1}{R} + 0.372(S - 3.0) \quad (14)$$

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:

$$\mathrm{SN} = a_1D_1 + a_2D_2 + \ldots \quad (15)$$

where

SN = structural number,
a_i = 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.

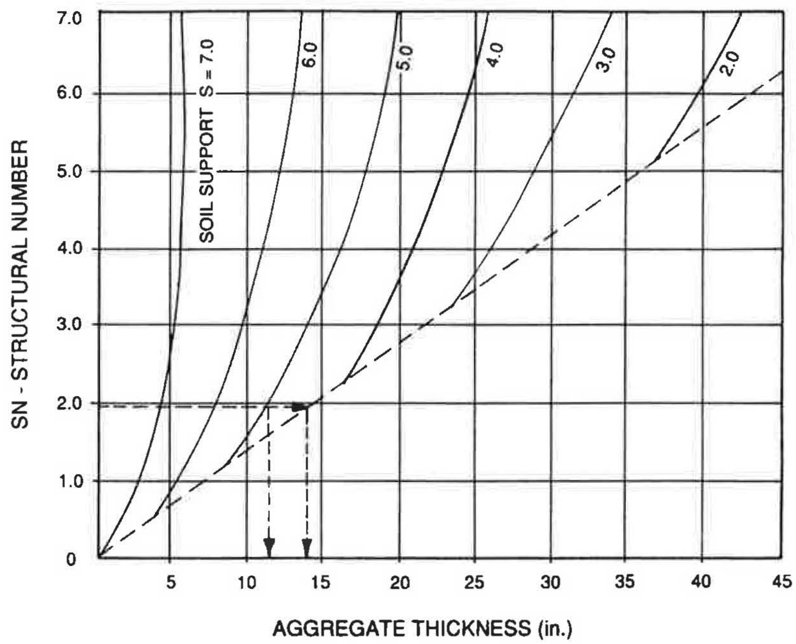

FIGURE 7 **Design chart for aggregate-surfaced pavements (*1*).**

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

$$t = (750/\mathrm{CBR})^{0.5} \tag{16}$$

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:

$$\mathrm{GL} = 0.162 + 0.0188(\mathrm{LT}) + 0.0382(\mathrm{F/C}) - 0.00110(\mathrm{TTU}) - 0.00213(\mathrm{P3/4}) \tag{17}$$

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

$$\mathrm{GL} = 0.1 + 0.01019(\mathrm{LT}) \tag{18}$$

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

$$\mathrm{GLIN} = \frac{B}{25.4}\,0.0045(\mathrm{LADT}) + \frac{3{,}381}{R} + 0.467(G) \tag{19}$$

where

GLIN = aggregate loss during time period considered (in.),
B = number of bladings during time period considered,

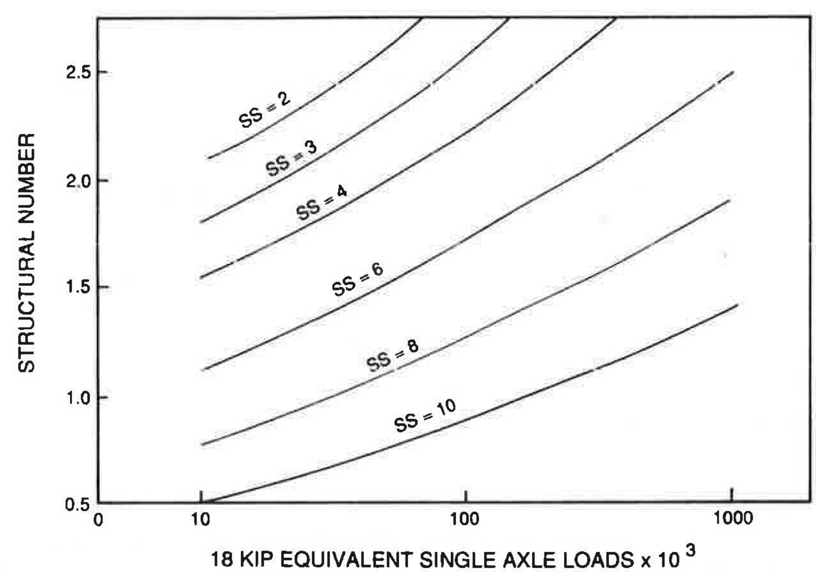

FIGURE 8 Structural number for various 18-kip equivalent axle loads and soil support values (*9*).

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:

$$\text{AGL} = f\left(\frac{T^2}{T^2 + 50}\right) * 4.2 + 0.092(T) + 0.0889(R^2) + 1.88(\text{VC}) \qquad (20)$$

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:

$$\text{GL} = D[\text{ADT}(0.047 + 0.0027 * N - 0.0005 * \text{P26}) - 0.365 * N - 0.0014 * \text{PF} + 0.048 * \text{P26}] \qquad (21)$$

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:

$$\text{GL} = (30 + 180 * \text{MMP} + 72 * \text{MMP} * G) * h * \text{ADT} * \Delta t * 10^{-5} \qquad (22)$$

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*)

		Rate of gravel loss (mm per 100,000 vehicle units) for gradient (%) and annual rainfall (mm/yr)				
		0%		3%		
Study Location	Units	1,000	2,000	1,000	2,000	Percentage heavy 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.

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.

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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.

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.

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TABLE 1 LENGTH OF CLASSIFIED ROAD NETWORK AS OF 1989 (*1*)

Class of road		Surface type: Bitumen	Gravel	Earth	TOTAL
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

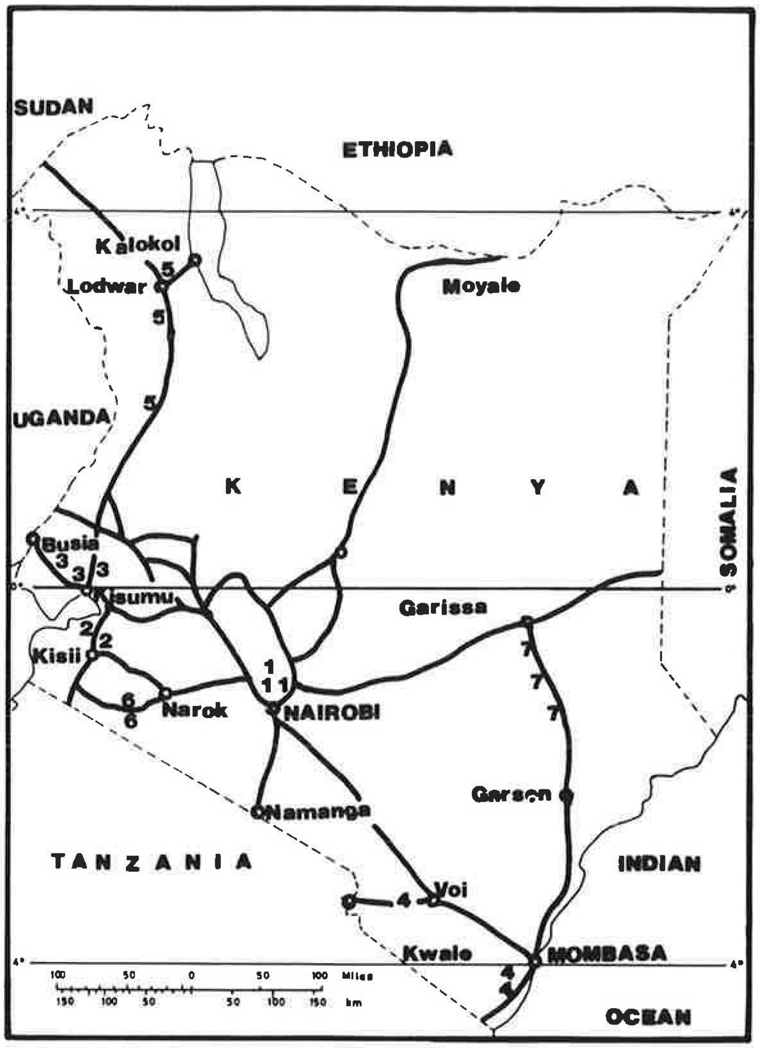

FIGURE 1 Location of low-cost trial sections.

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.

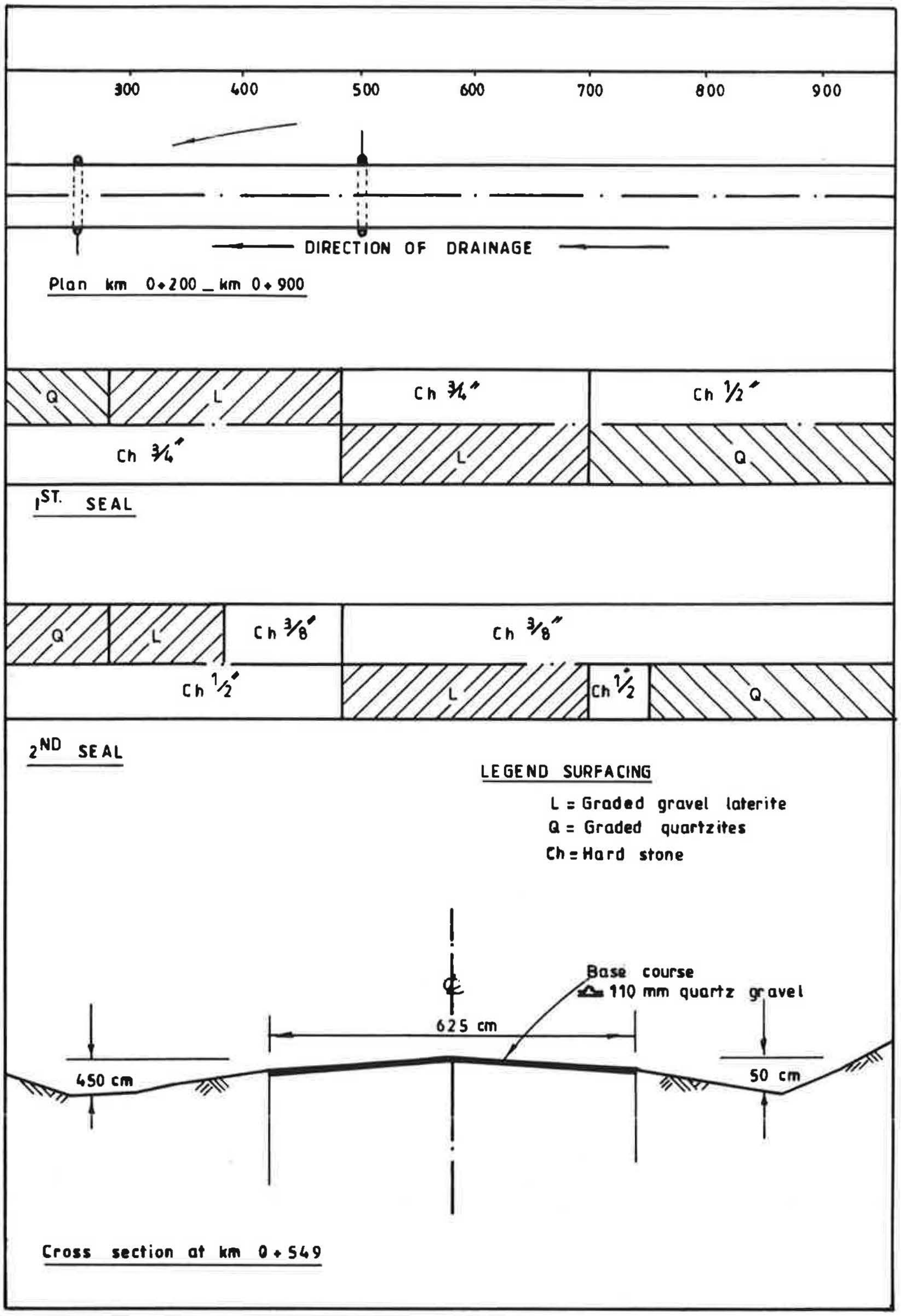

FIGURE 2 Details of D-291 Majengo trial section.

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

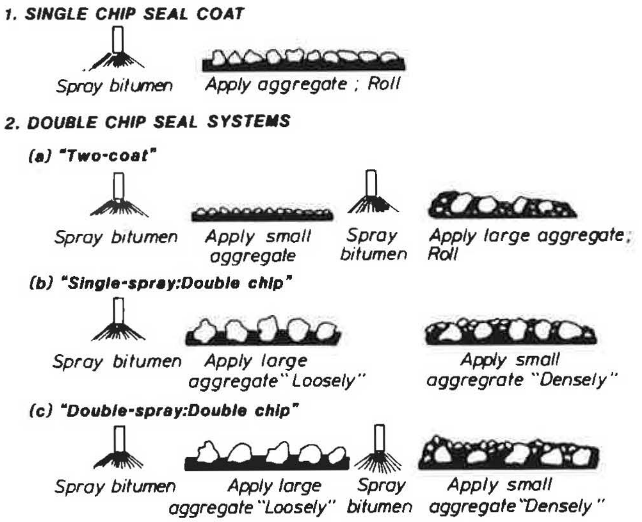

FIGURE 1 Types of seal coat systems (*7*).

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.

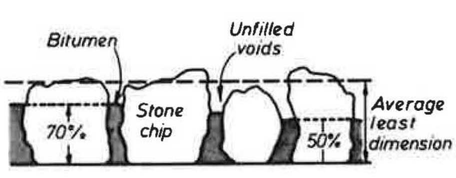

FIGURE 2 Cross section of a seal coat system.

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

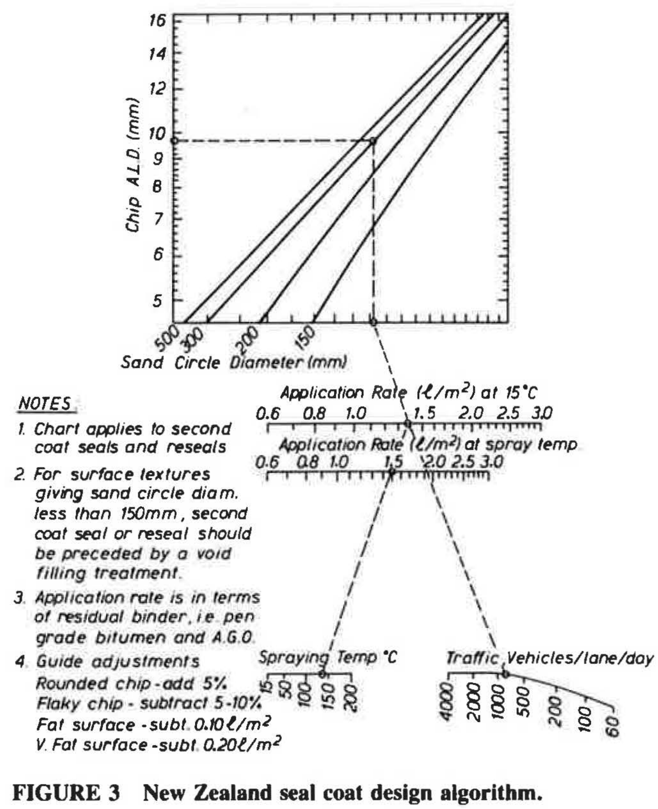

FIGURE 3 New Zealand seal coat design algorithm.

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 comprehensive 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.

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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 buildup 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.

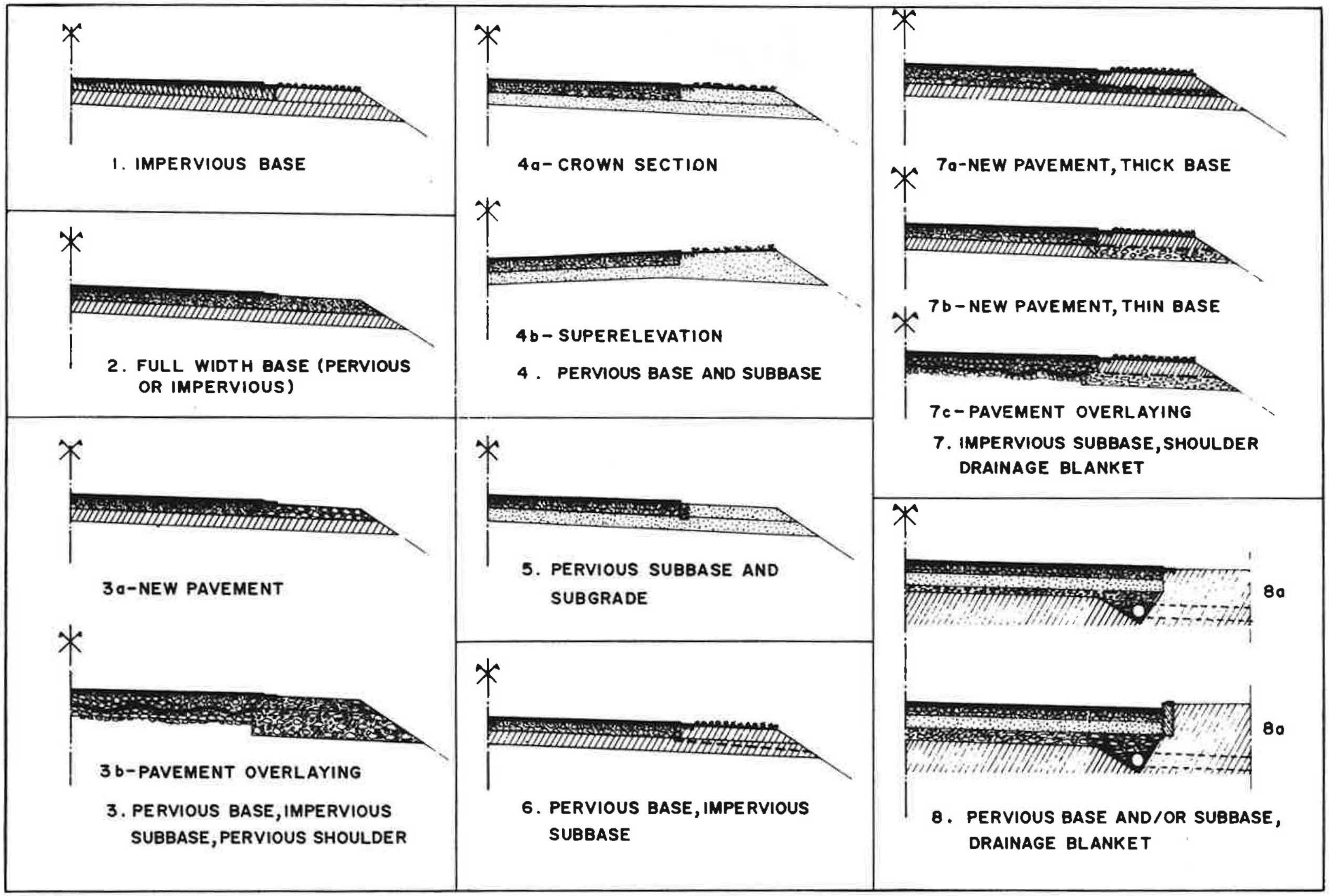

FIGURE 4 Catalog of typical pavement cross sections, Madagascar, 1986. Impervious pavement materials are represented by hatching. Pervious subbase (and shoulder) materials are most often sands and represented by dotting. Coarse materials in bases, shoulders, and blanket layers as well as turf surfacing 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 Width(m)	Base course Materials	Shoulders	Subdrainage		Construction cost (%)
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 intermediate 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

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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

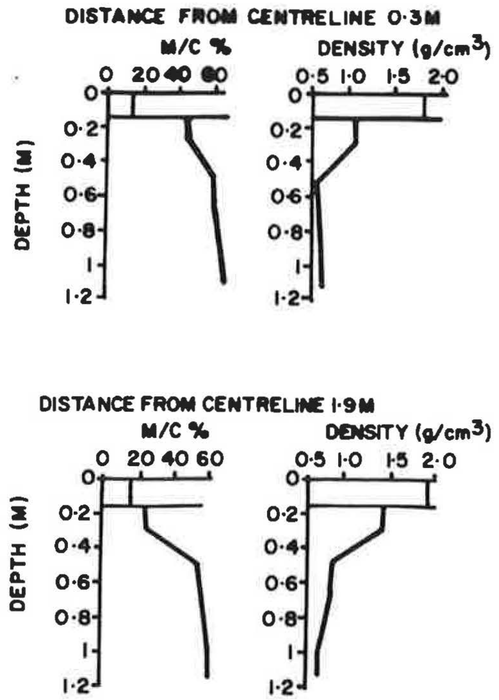

FIGURE 1 Moisture and density variation with depth for ES10 (Gatura II).

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;

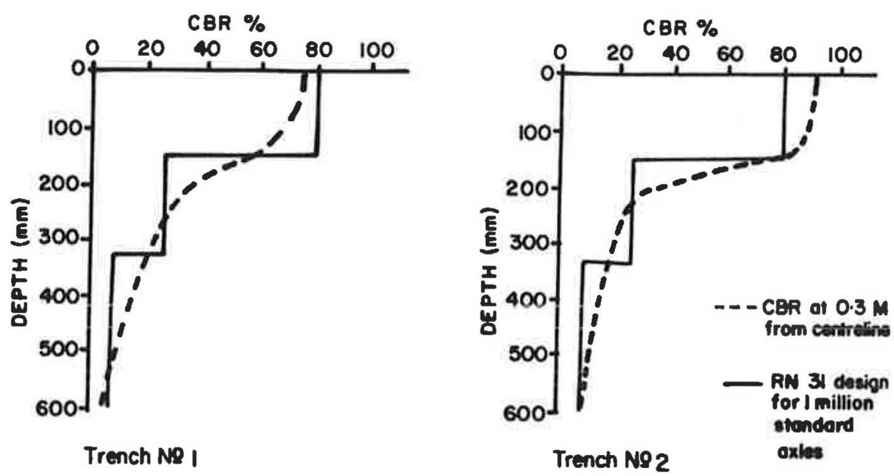

FIGURE 2 Variation of CBR with depth below surface for ES10.

- 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/yd^2 of MC-250 asphalt for coarse aggregate or 0.35 gal/yd^2 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/yd^2 of CRS-2 asphalt.
6. Immediately apply 20 to 30 lb/yd^2 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/yd^2 of CRS-2 asphalt (preferably with latex additive).
11. Immediately apply 15 to 25 lb/yd^2 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/yd^2 (0.40 gal/yd^2 penetration application + 0.55 gal/yd^2); typical aggregate quantity is 50 lb/yd^2; and typical cost is \$2.05/yd^2 (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/yd^2).
4. Chip/blot immediately with ⅜-in. chips at 18 to 25 lb/yd^2. 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/yd^2 of CRS-1 or CRS-2 asphalt with latex additive.
9. Chip again with 18 to 25 lb/yd^2 of ⅜- or ¼-in. chips.
10. Rubber tire roll again.

Typical asphalt quantity is 0.70 gal/yd^2; typical aggregate quantity is 45 lb/yd^2; and typical cost is \$1.75/yd^2 (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

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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 (wheel-path) 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

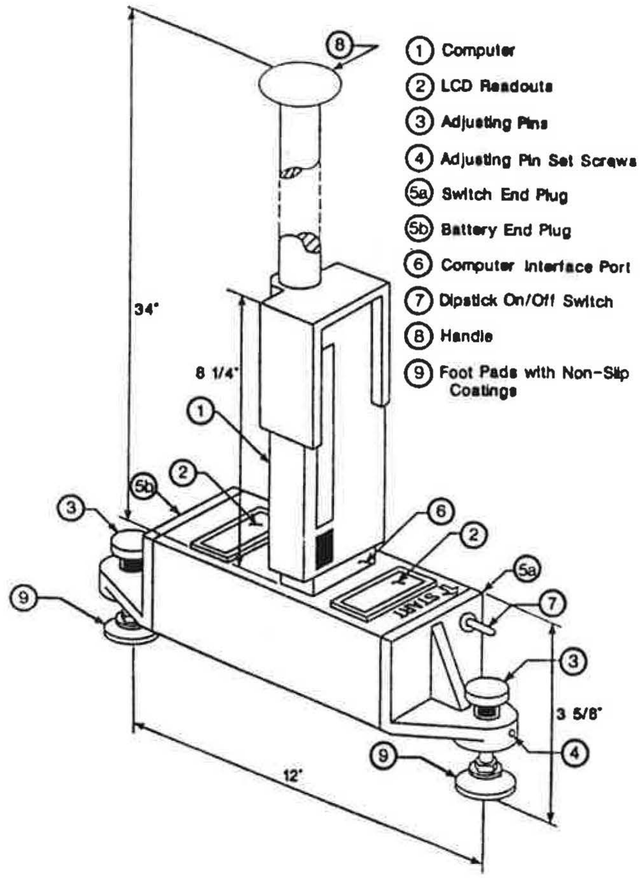

FIGURE 1 Auto-read version of the dipstick.

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

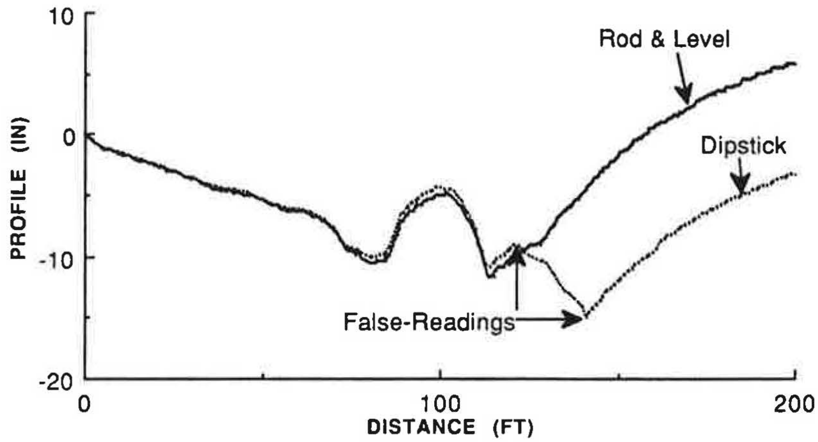

FIGURE 2 Dipstick data with false readings versus rod-and-level survey.

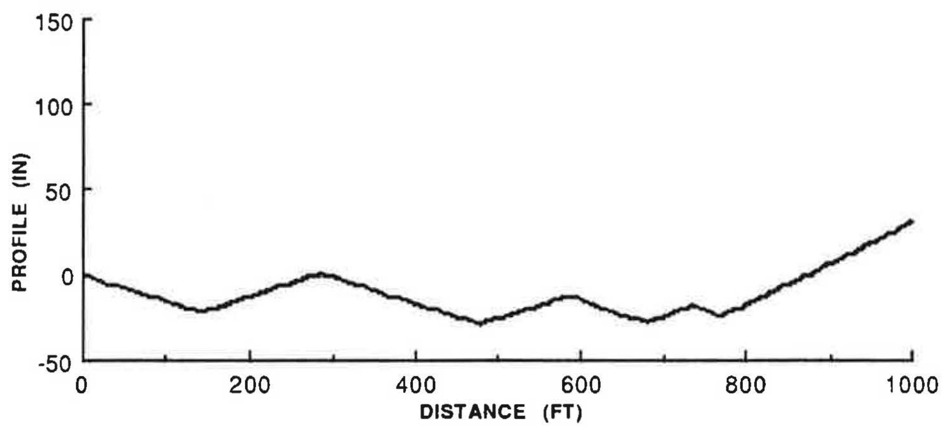

FIGURE 3 Dipstick data with false readings.

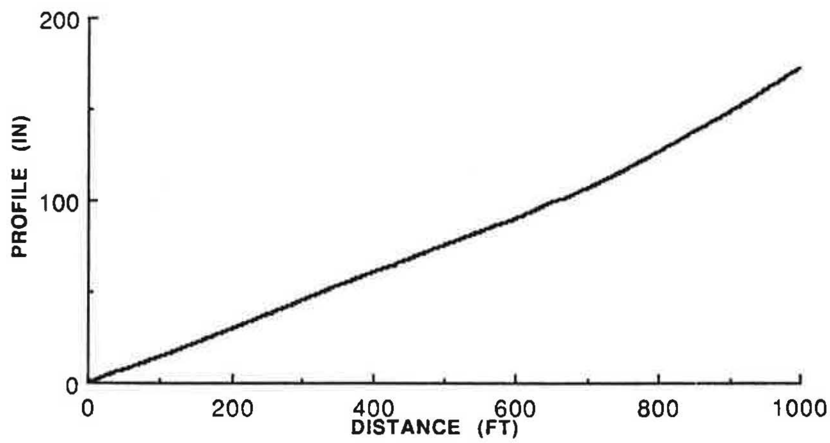

FIGURE 4 Profile of surface shown in Figure 3 with proper slope.

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 disadvantages 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

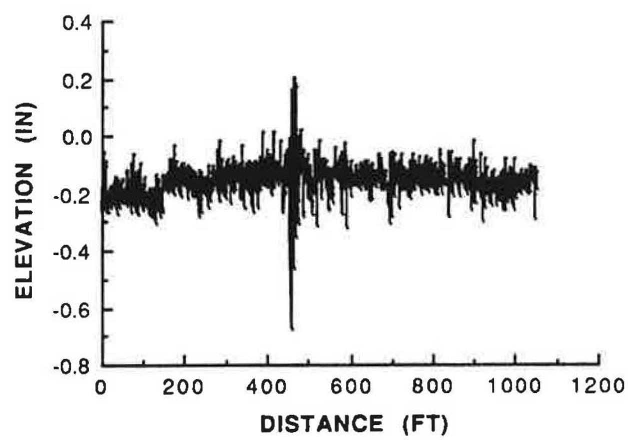

FIGURE 5 Dipstick elevations from ATS 36 outside wheel path.

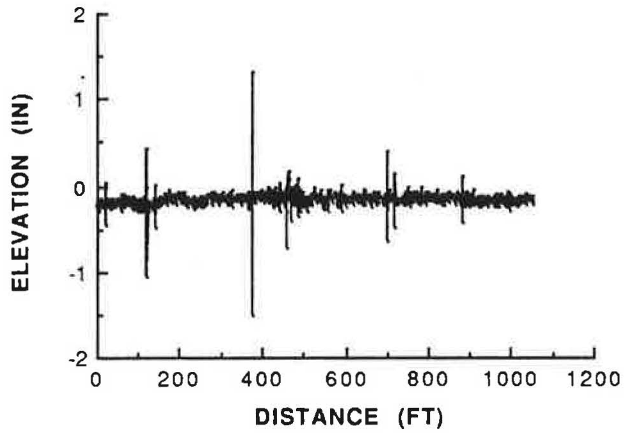

FIGURE 6 Rod-and-level elevations of ATS 36 outside wheel path before corrections.

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.

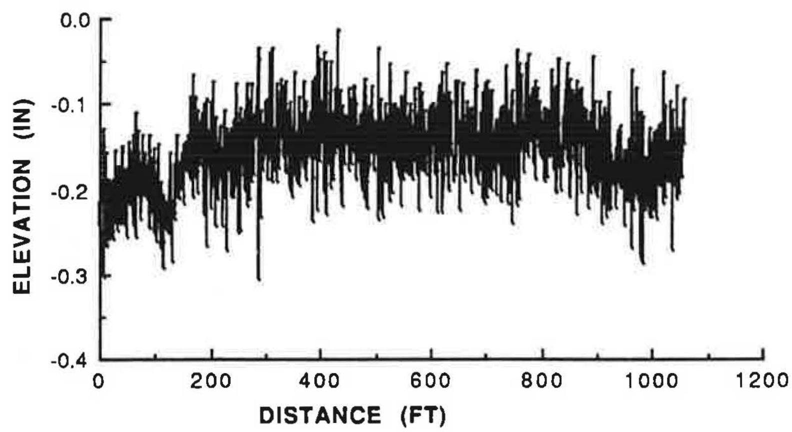

FIGURE 7 Dipstick elevations from ATS 36 inside wheel path.

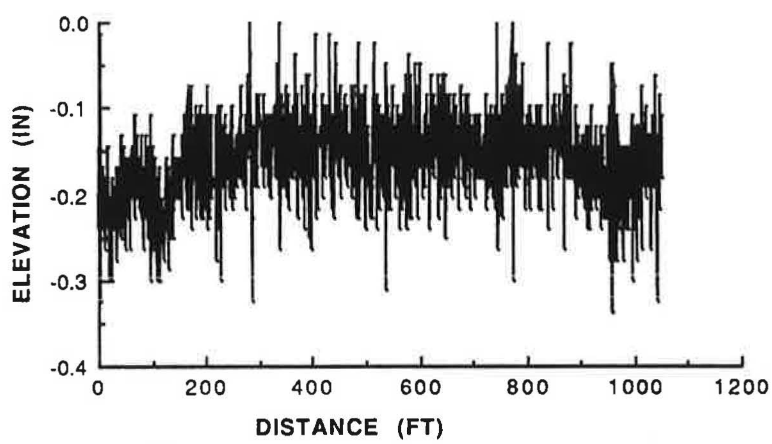

FIGURE 8 Rod-and-level elevations from ATS 36 inside wheel path.

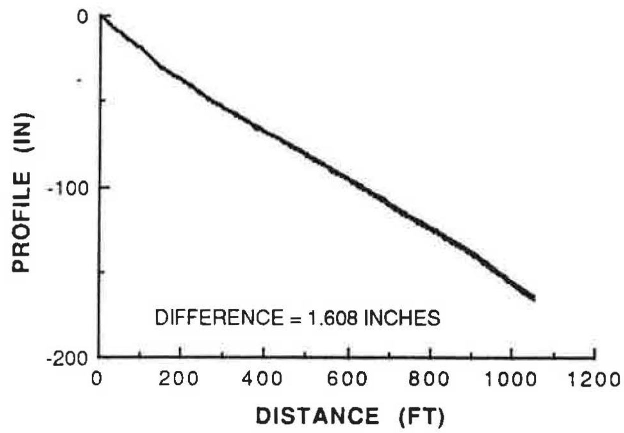

FIGURE 9 Profile comparison of dipstick and rod-and-level on outside wheel path of ATS 36.

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;

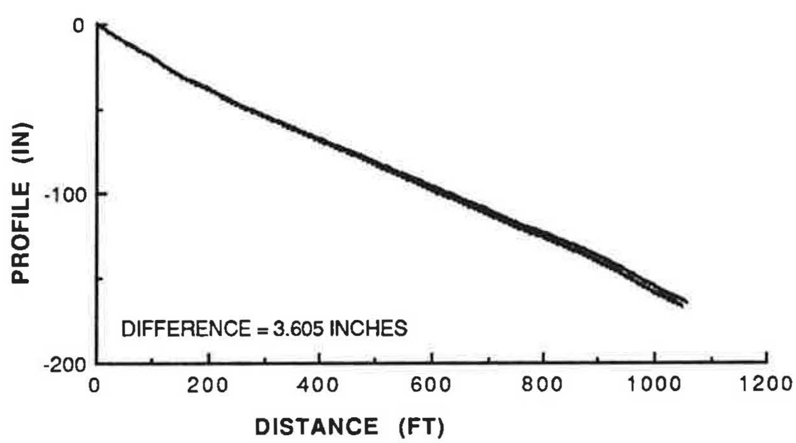

FIGURE 10 Profile comparison of dipstick and rod-and-level on inside wheel path of ATS 36.

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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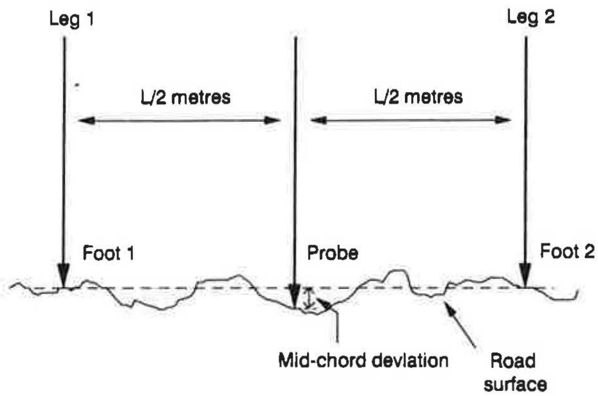

FIGURE 1 Measurement of midchord deviation.

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_r 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

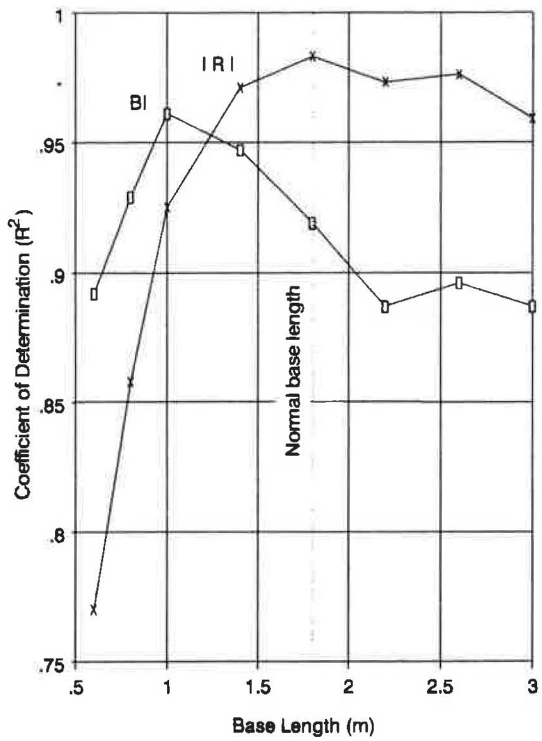

FIGURE 2 Roughness measuring accuracy for different base lengths.

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.

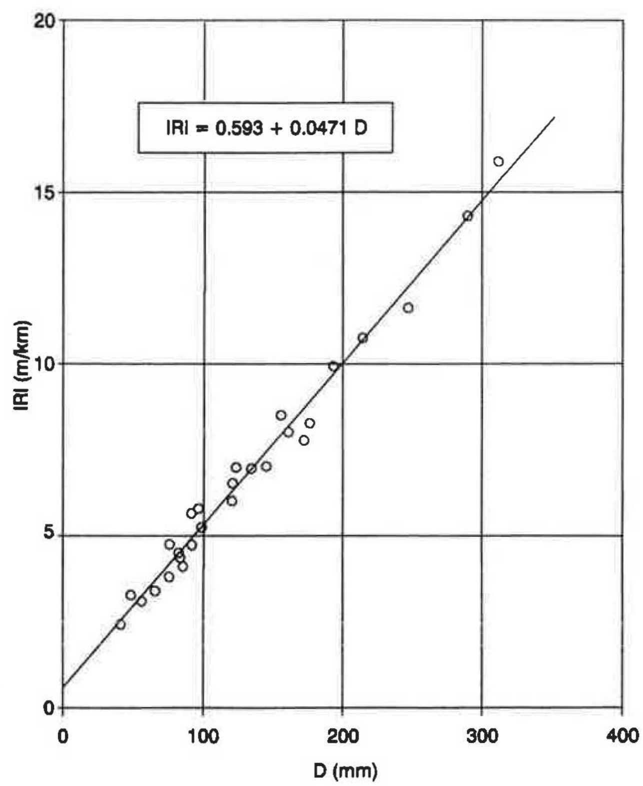

FIGURE 3 Relationship between IRI and *D*.

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.

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).

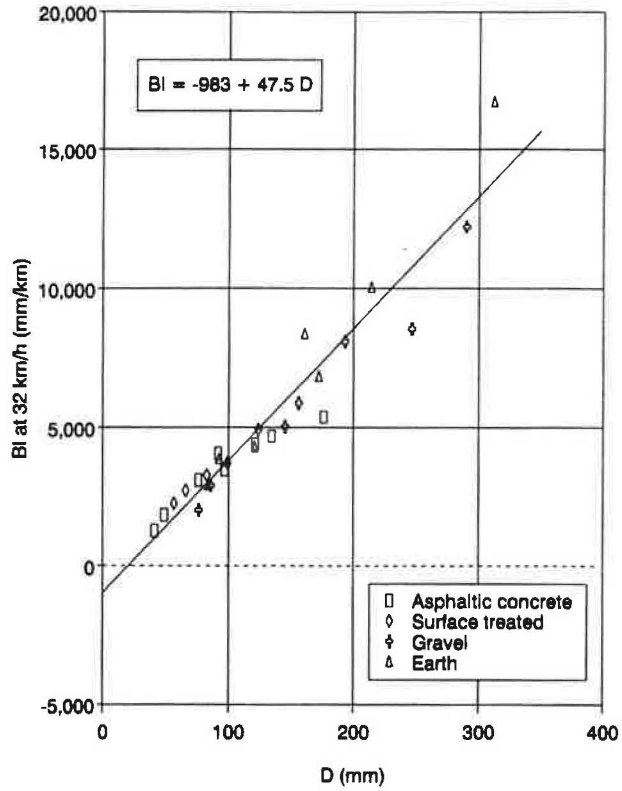

FIGURE 4 Relationship between BI and *D*.

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

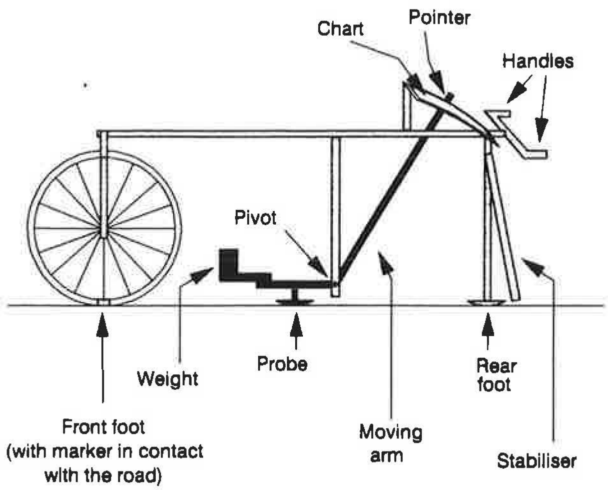

FIGURE 5 Sketch of the MERLIN.

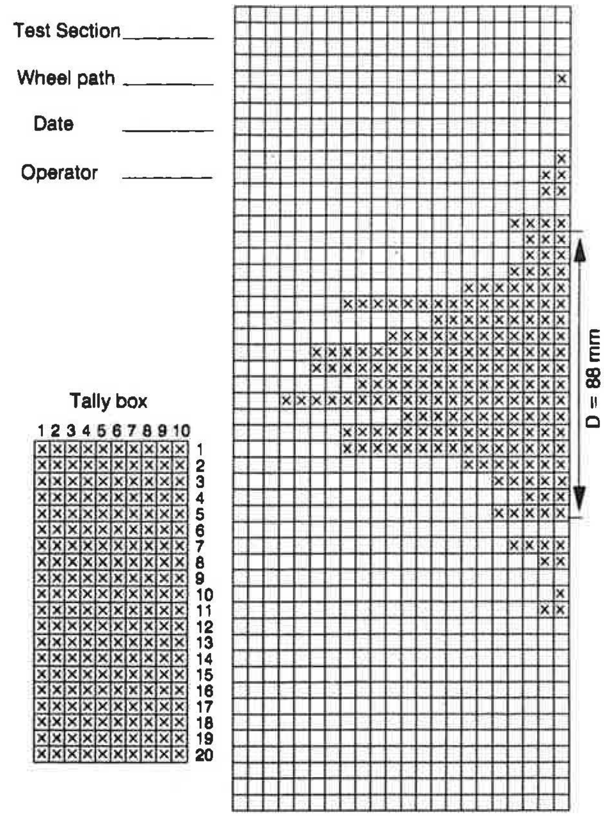

FIGURE 6 Specimen result.

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

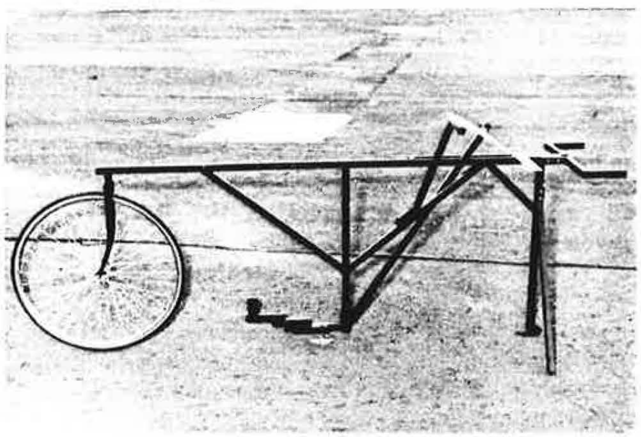

FIGURE 7 The MERLIN.

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.

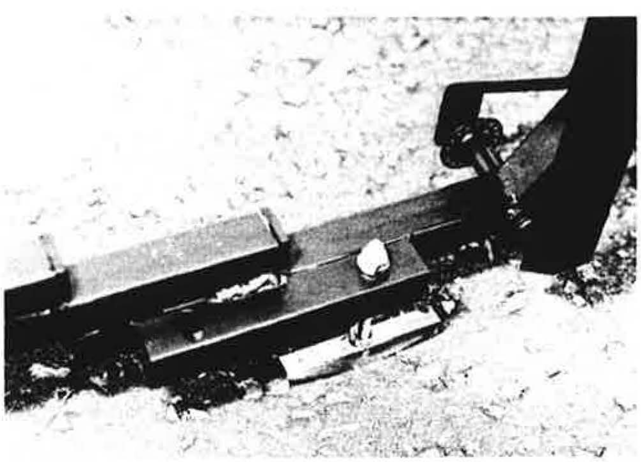

FIGURE 8 The probe and moving arm.

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

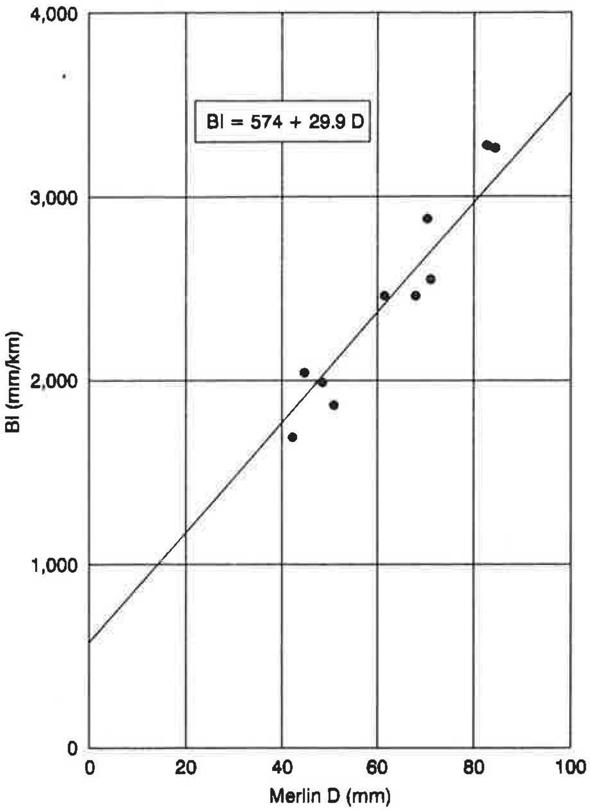

FIGURE 9 Calibration check on AC.

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

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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	Average weighted branch URCI 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 1988	1989	No. of branches with average weighted URCI 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 1988	1989	No. of branches with average weighted URCI 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	Average weighted branch URCI 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.

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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.
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.

*Note: Individual roadways may not have all of the types of distress listed for any particular rating. They may have one or two types.

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. Little if any ditching. Filled or damaged culverts. Severe rutting (over 3" deep), over 25% of area. Severe potholes (over 4" deep), over 25% of area. Many areas (over 25%) with little or no aggregate.	Travel is difficult and road may be closed at times. Needs complete rebuilding and/or new culverts.

*Note: Individual roadways may not have all of the types of distress listed for any particular rating. They may have one or two types.

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

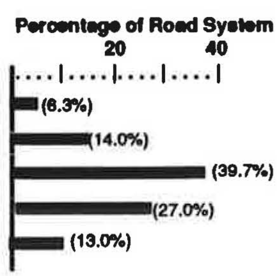

FIGURE 2 Surface condition ratings.

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

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Observed Behavior of Bituminous-Surfaced Low-Volume Laterite Pavements

Cesar Queiroz, Steve Carapetis, Henry Grace, and William Paterson

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

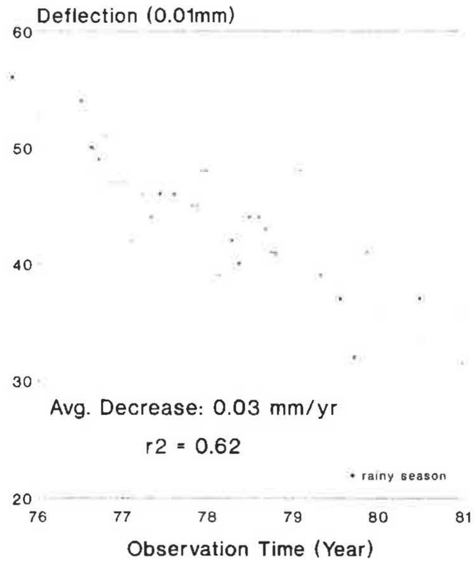

FIGURE 1 Section 001 deflection history.

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	

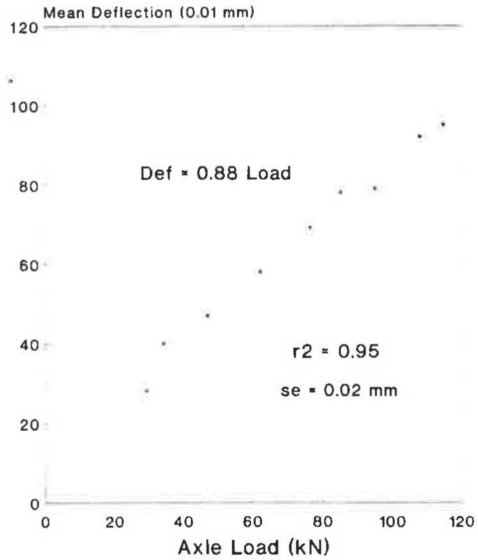

FIGURE 2 Deflection versus load, Section 006.

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:

$$B = 23.4D \tag{1}$$

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):

$$B = 35.3D - 4.3D^2 \tag{2}$$

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:

$$B = 35.2D \tag{3}$$

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

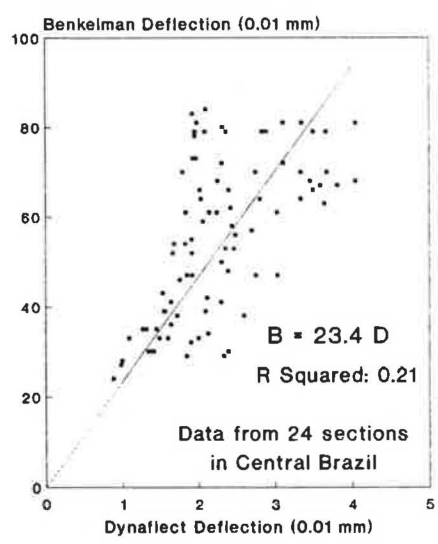

FIGURE 3 Relationship between Benkelman beam and Dynaflect deflections on laterite pavements in Brazil.

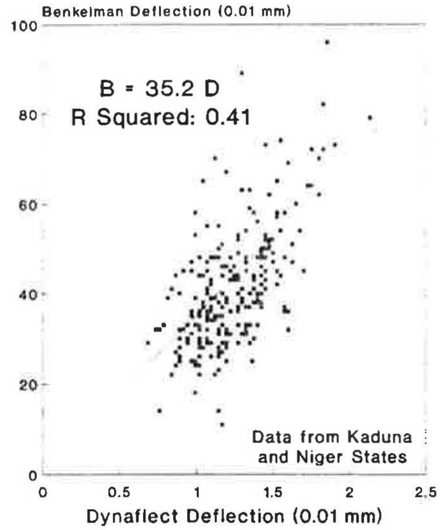

FIGURE 4 Relationship between Benkelman beam and Dynaflect deflections on laterite pavements in Nigeria.

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):

$$\text{SCBR} = 0.78\text{UCBR} \tag{4}$$

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:

$$\text{ICBR} = 1.02\text{UCBR} \tag{5}$$

and

$$\text{ICBR} = 1.27\text{SCBR} \tag{6}$$

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*).

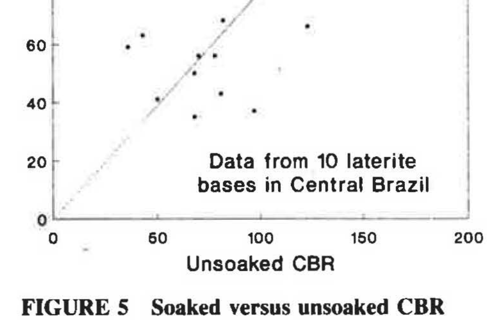

FIGURE 5 Soaked versus unsoaked CBR values.

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:

$$\begin{aligned} \mathrm{GL} = D[&\mathrm{ADT}(0.059 + 0.0027N - 0.0006\ \mathrm{P26}) \\ &- 0.367(\mathrm{N}) - 0.0014(\mathrm{PF}) + 0.0474(\mathrm{P26})] \end{aligned} \quad (1)$$

[$r^2 = 0.84$; 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:

$$\mathrm{GL} = D[1.58 + 0.366(G) + 0.083(\mathrm{SV}) - 0.210(\mathrm{PI}) + 0.0132(\mathrm{NC}) + 0.0081(\mathrm{NT}) + 420.45/R] \quad (2)$$

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:

$$\mathrm{MLA} = 3.65[3.46 + 0.246(\mathrm{MMP})(\mathrm{RF}) + (\mathrm{KT})(\mathrm{ADT})] \quad (3)$$

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:

$$\ln \mathrm{R} = D[-13.8 + 0.00022(\mathrm{PF}) + 0.064(\mathrm{S1}) + 0.137(\mathrm{P26}) + 0.0003(N)(\mathrm{ADT}) + \mathrm{GM}(6.42 - 0.063\ \mathrm{P26})] \quad (4)$$

($r^2 = 0.22$; $n = 7005$; 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:

$$\mathrm{LDQ} = D\{0.4314 - 0.1705(\mathrm{T2}) + 0.001159(\mathrm{NC}) + 0.000895(\mathrm{NT}) - 0.000227(\mathrm{NT})(G) + S[-0.1442 - 0.0198(G) + 0.00621(\mathrm{SV}) - 0.0142(\mathrm{PI}) - 0.000617(\mathrm{NC})]\} \quad (5)$$

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:

$$\mathrm{QI}(\mathrm{TG}_2) = \mathrm{QIMAX}_j - b[\mathrm{QIMAX}_j - \mathrm{QI}(\mathrm{TG}_1)] \quad (6)$$

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

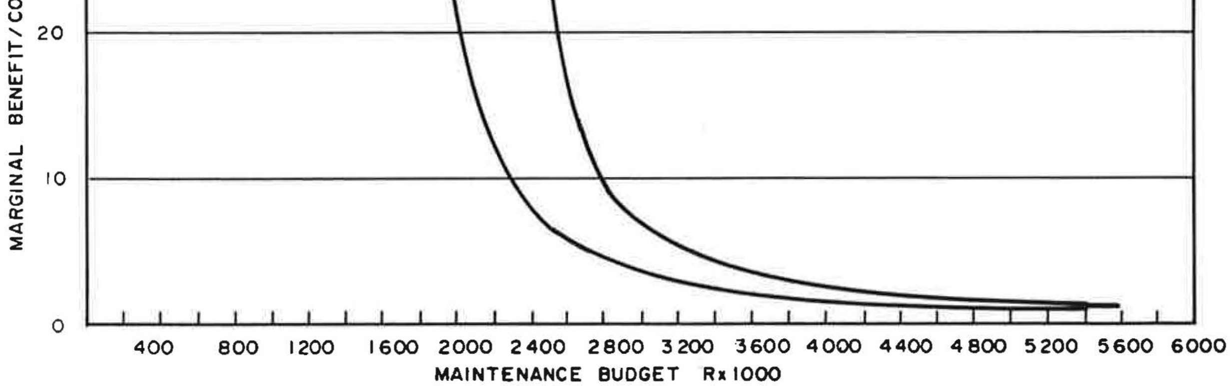

FIGURE 1 Graph for determining the appropriate maintenance budget.

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

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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

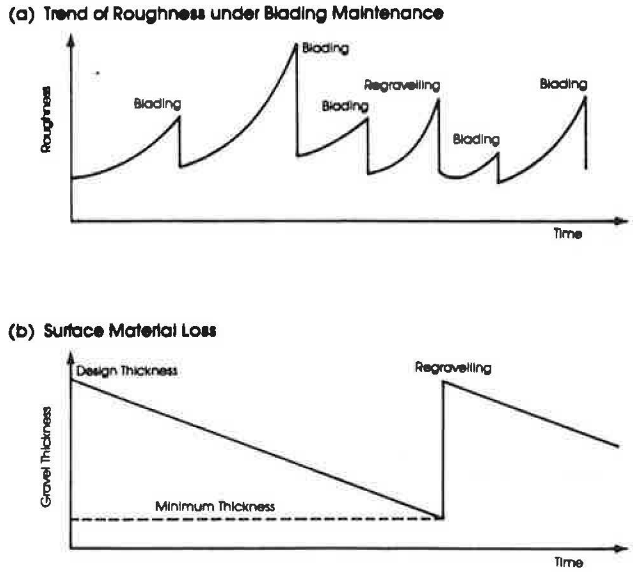


FIGURE 1 Trends of major deterioration modes of unpaved roads under maintenance.

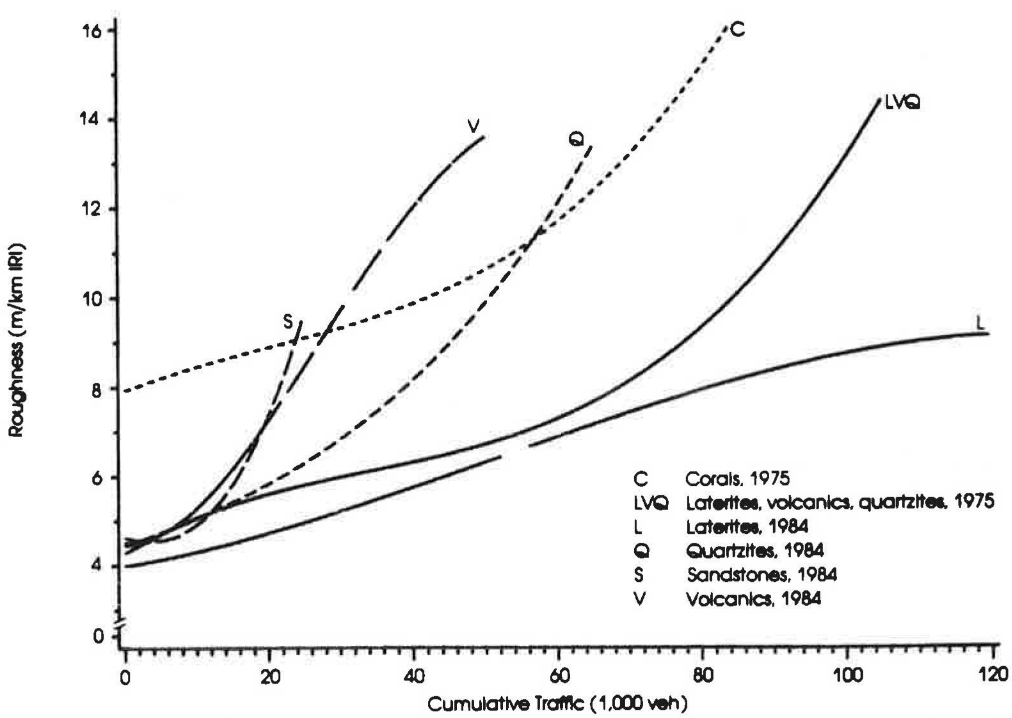
observed for different materials and between studies for a similar material. The result of blading was modeled as achieving a constant level of roughness, regardless of previous conditions. This level was 4.3 m/km on the International Roughness Index (IRI) for lateritic, quartzitic, and volcanic gravels, 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.



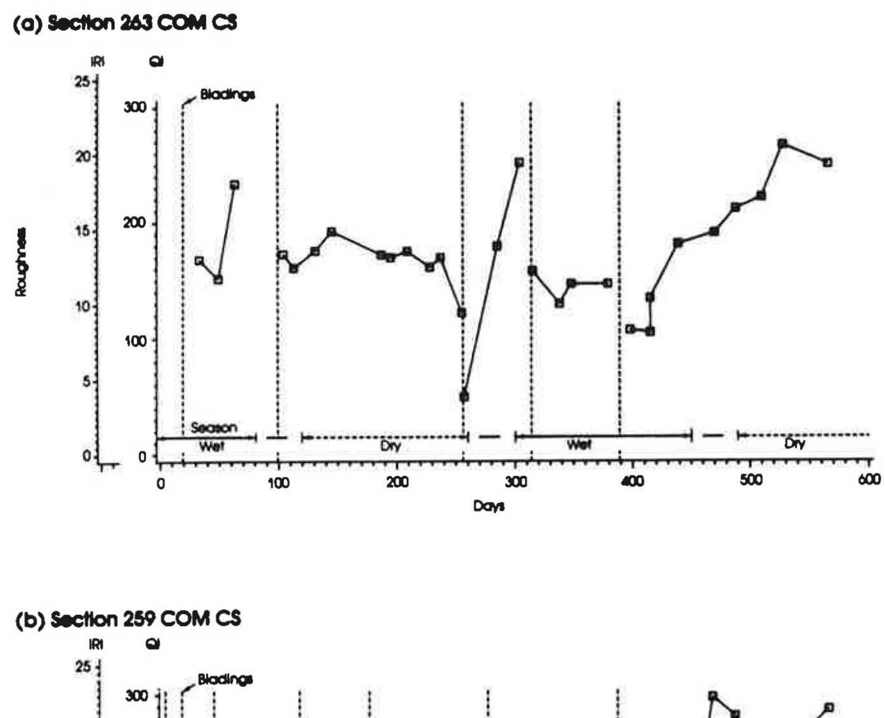
Note: Roughness conversion: $RI = 0.0032\ RBI^{0.89}$ where RBI = mm/km TRRL Bump Integrator Trailer, and RI = m/km IRI.

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:

$$p = \exp\{-0.001[0.461 + 0.0174(\mathrm{ADL}) + 0.0114(\mathrm{ADH}) - 0.0287(\mathrm{ADT})(\mathrm{MMP})]T\} \quad (4)$$

$$RG_{max} = 21.4 - 32.4(0.5 - \mathrm{MGD})^2 + 0.97\mathrm{KCV} - 7.64G\mathrm{MMP}$$

$$\text{and } RG_{max} \geq 12 \quad (5)$$

$$q = 0.553 + 0.230\ \mathrm{MGD} \quad (6)$$

$$RG_{min} = 0.361\mathrm{D95}[1 - 2.78(\mathrm{MG}')]$$

$$0.8 < RG_{min} < 8 \quad (7)$$

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}($1\ \mathrm{km}^{-1} = 180\pi$ 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*).

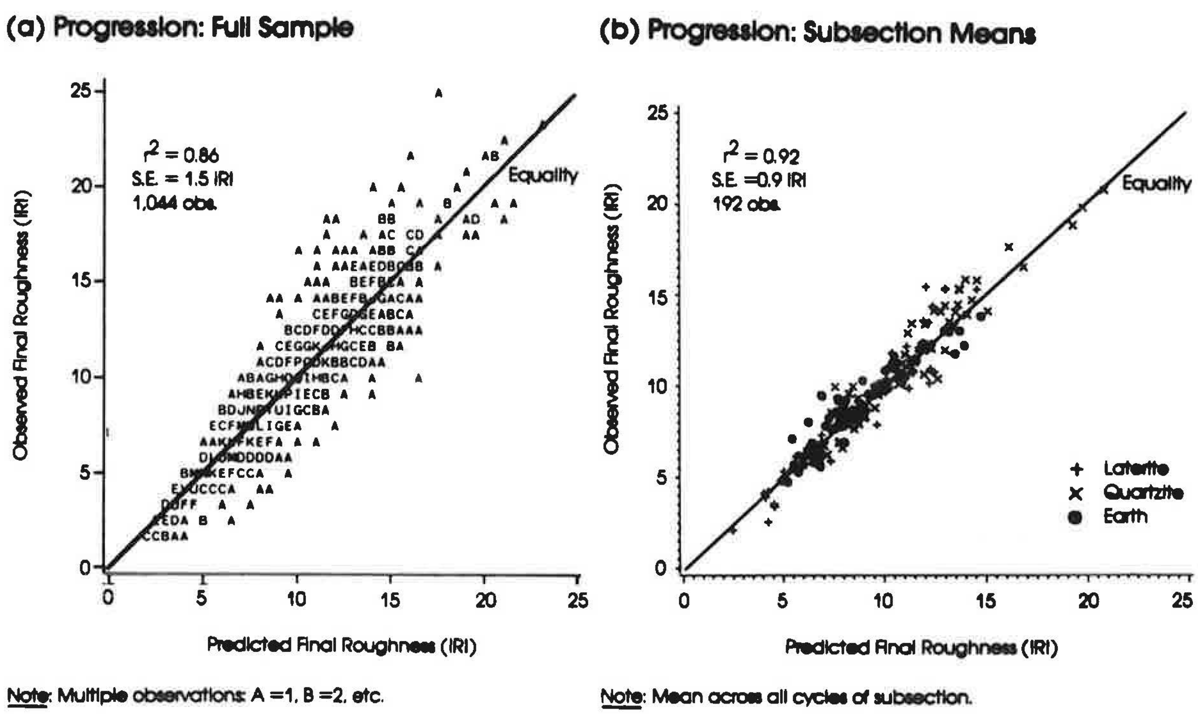


FIGURE 4 Goodness of fit of model for roughness progression on one road, with and without the between-cycle variance.

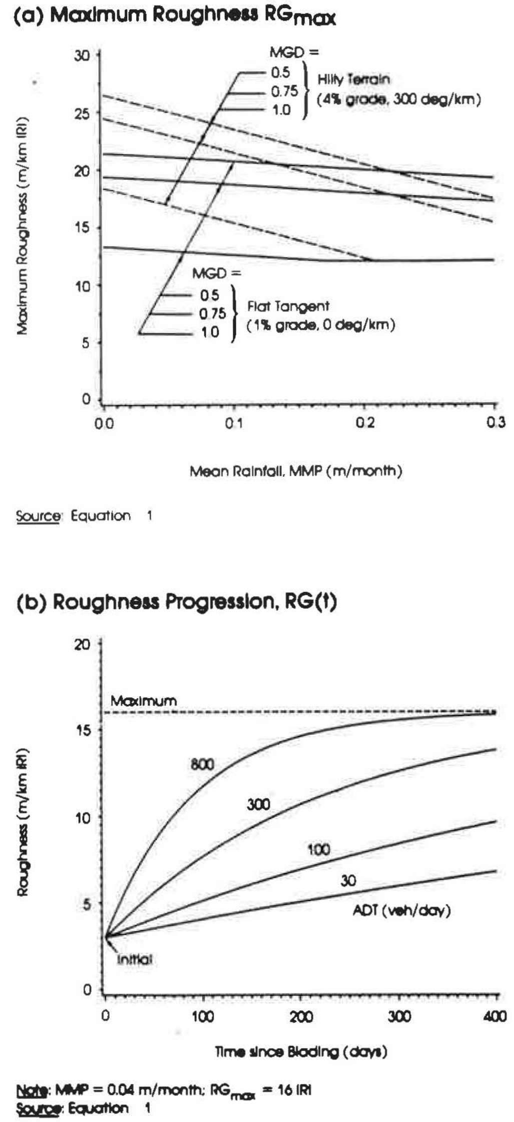


FIGURE 5 Predictions of maximum roughness and roughness progression rates relative to traffic volumes.

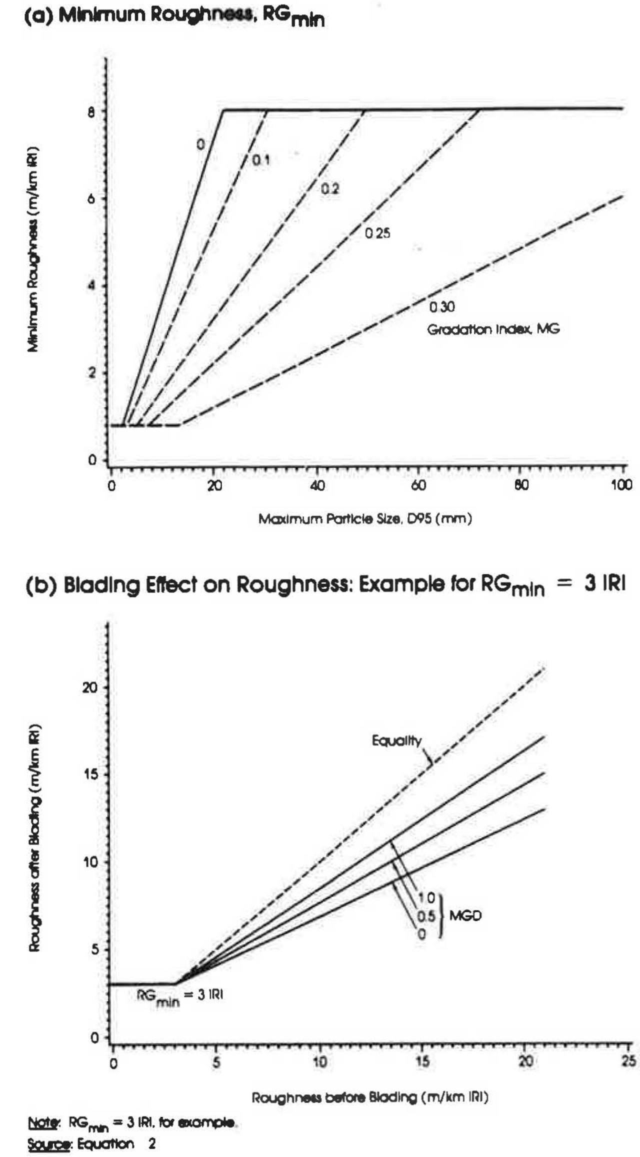


FIGURE 6 Predictions of minimum roughness and blading effects.

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

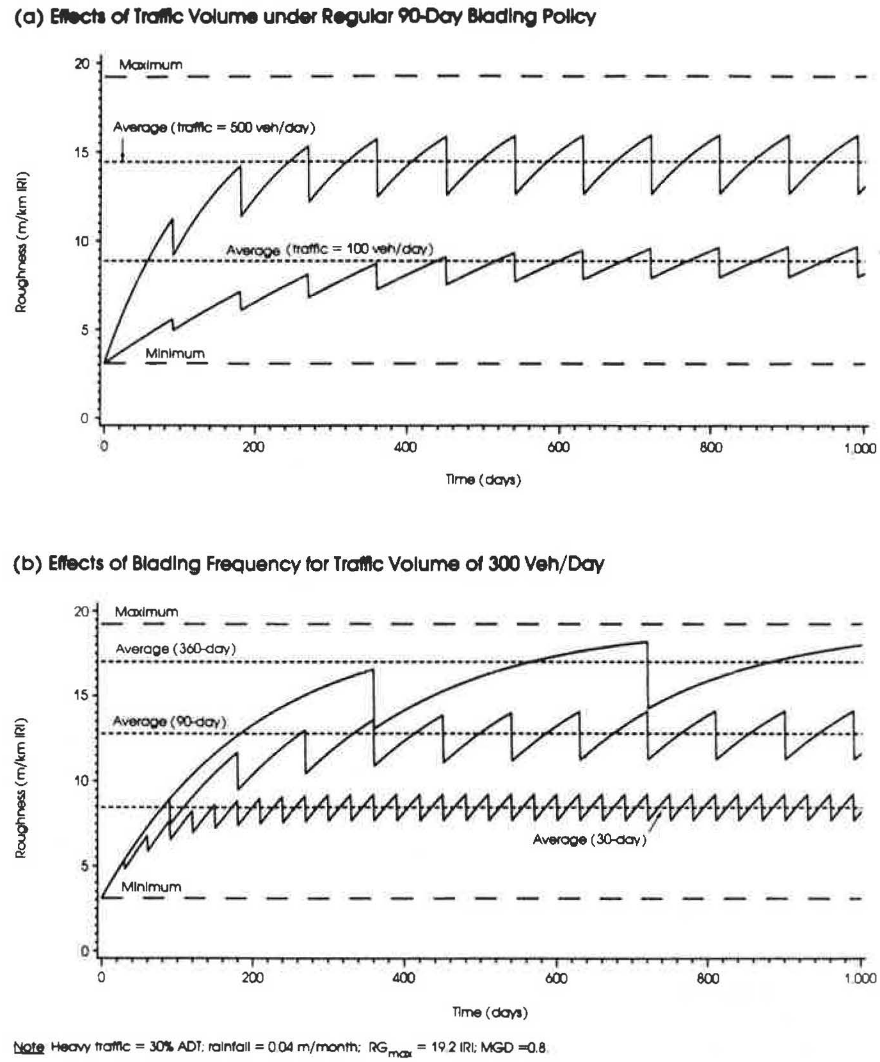
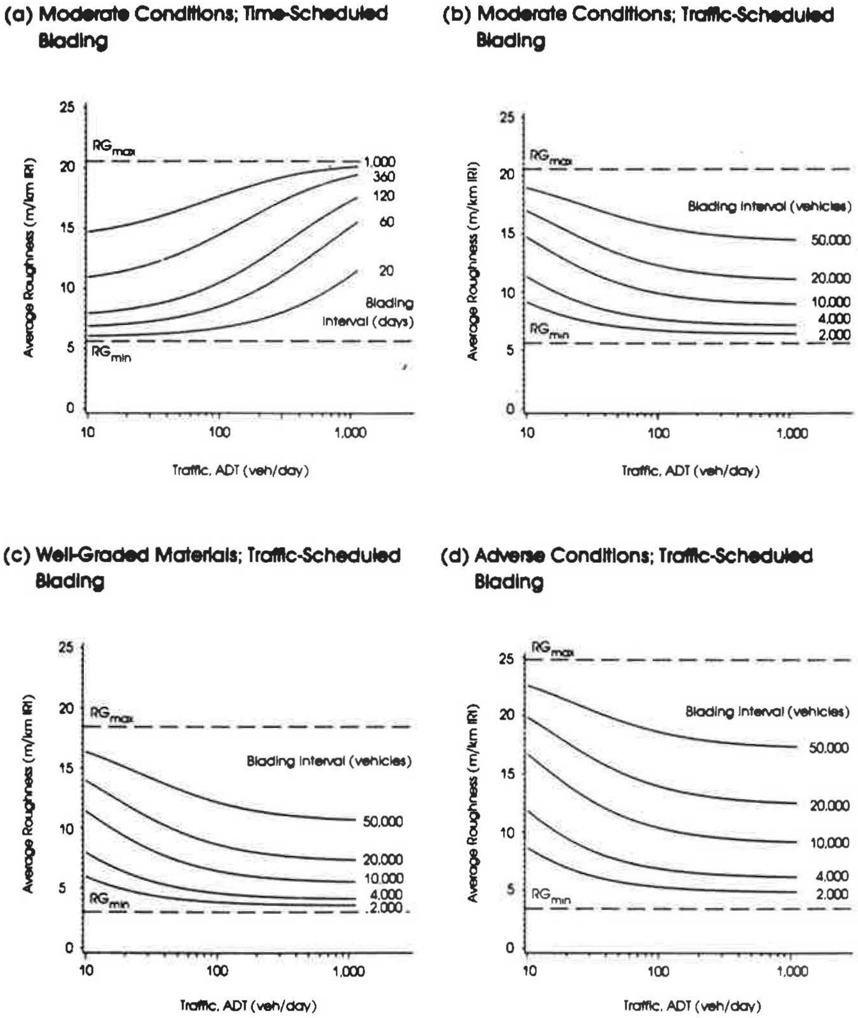


FIGURE 7 Predictions of roughness trends under regular maintenance.

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.

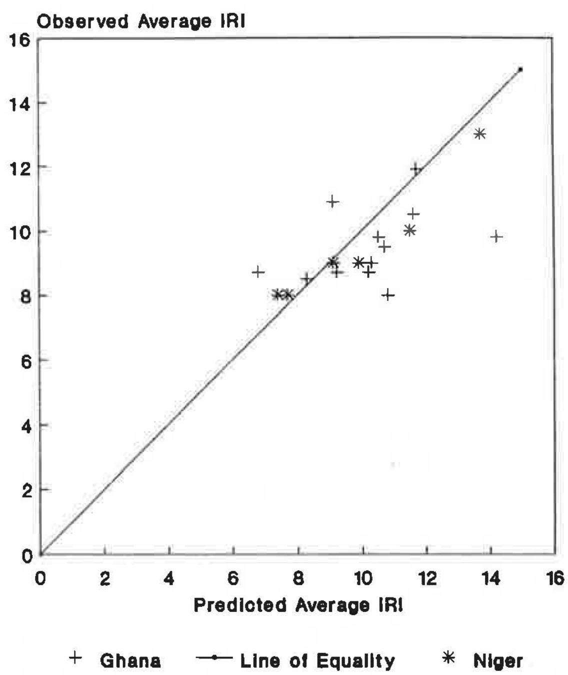


FIGURE 9 Comparison of observed and predicted average roughness and progression rates for Ghana and Niger.

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:

$$\text{MLA} = 3.65[3.46 + 2.46(\text{MMP})(G) + (\text{KT})(\text{ADT})] \tag{8}$$

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

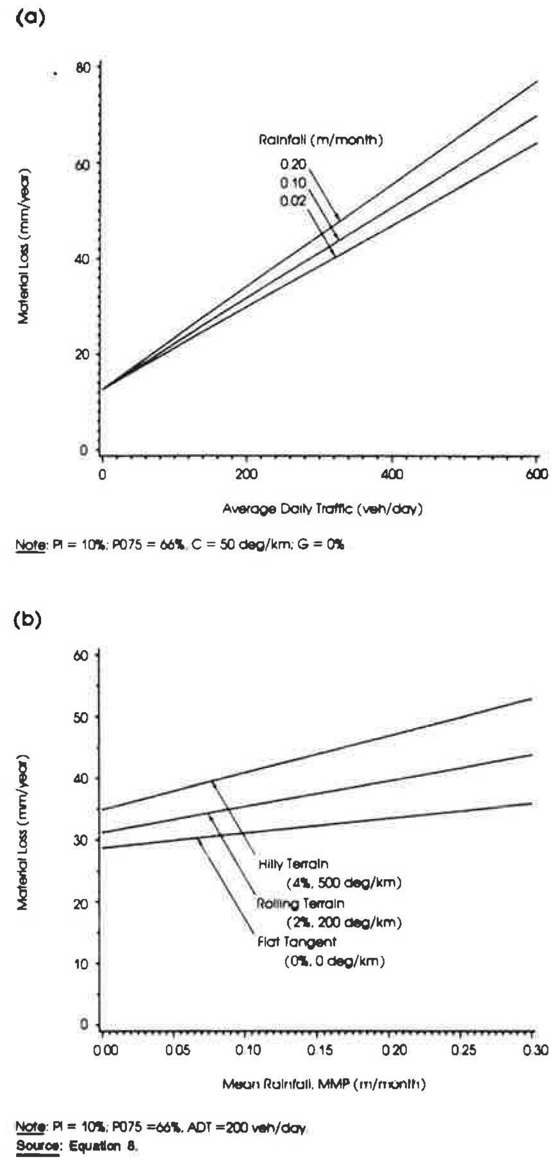


FIGURE 10 Predictions of surface material loss related to traffic, terrain, and rainfall: *a*, effects of traffic and rainfall on flat terrain; *b*, effects of terrain and rainfall under 200 veh/day.

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 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.

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.

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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

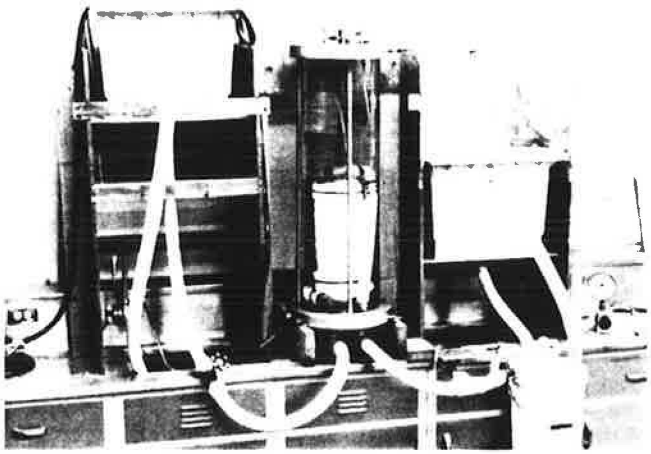

FIGURE 1 Laboratory test system.

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

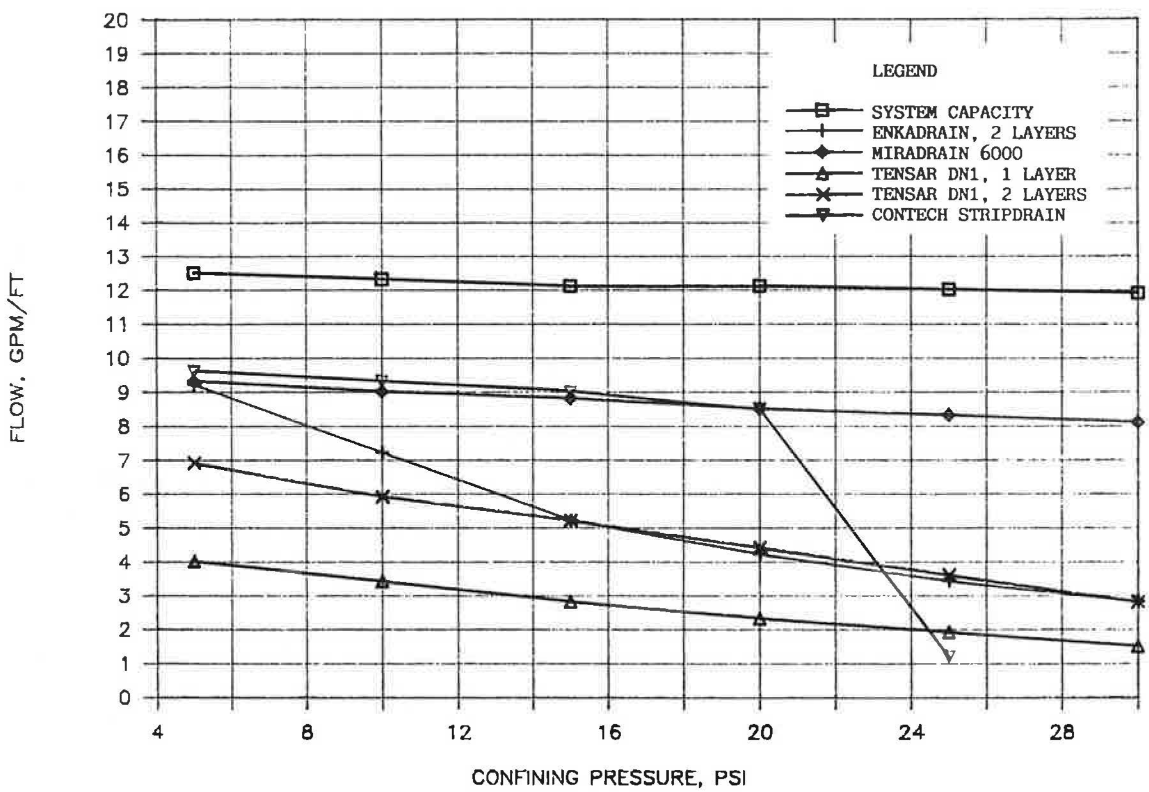

FIGURE 2 Equilibrium flow versus confining pressure: 1.0 gradient.

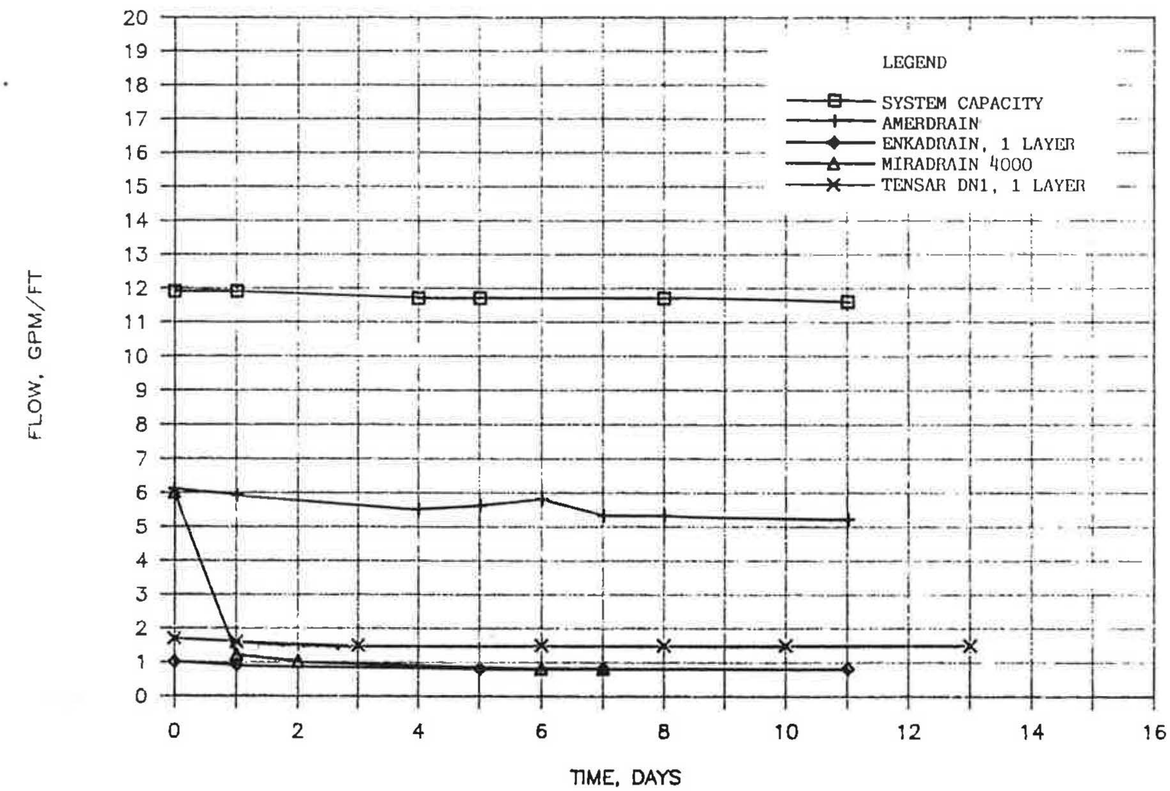

FIGURE 3 Equilibrium flow versus time (0–16 days, 30-psi confining pressure, 1.0 gradient): system capacity; Amerdrain; Enkadrain, one layer; Miradrain 4000; Tensar DN1, one layer.

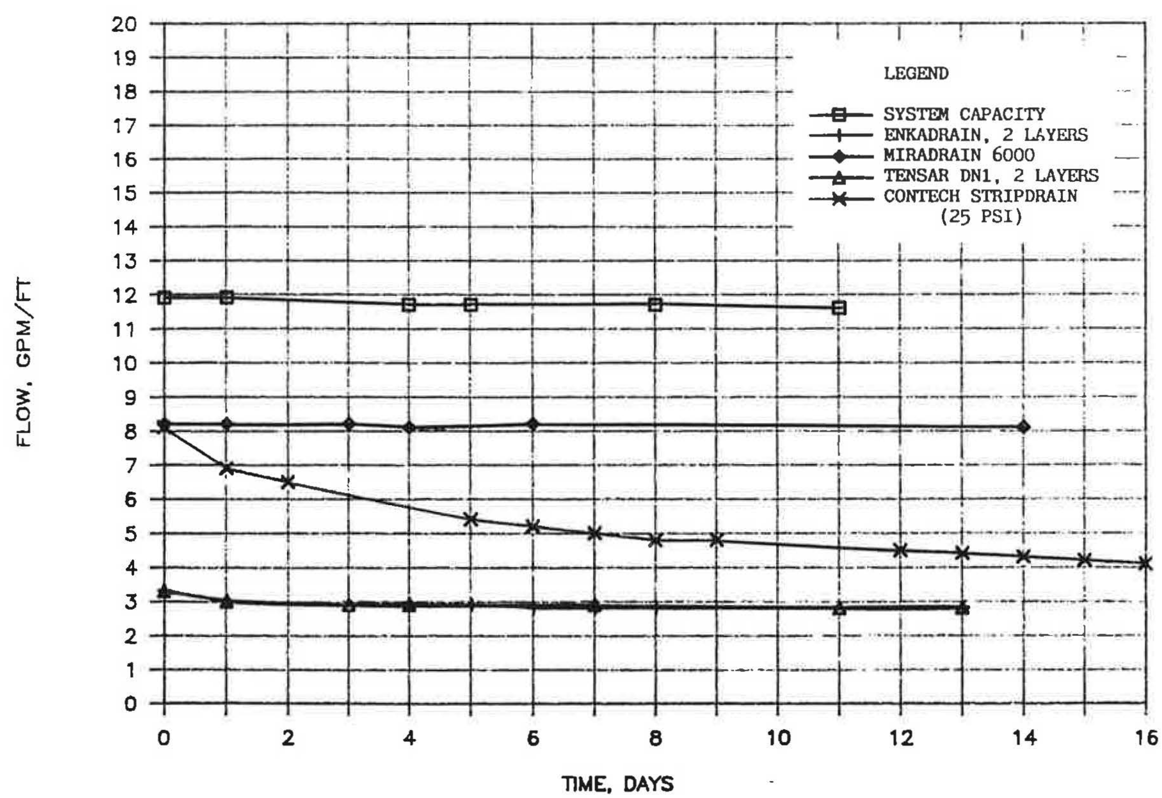

FIGURE 4 Equilibrium flow versus time (0–16 days, 30-psi confining pressure, 1.0 gradient): system capacity; Enkadrain, two layers; Miradrain 6000; Tensar DN1, two layers; Contech Stripdrain (25 psi).

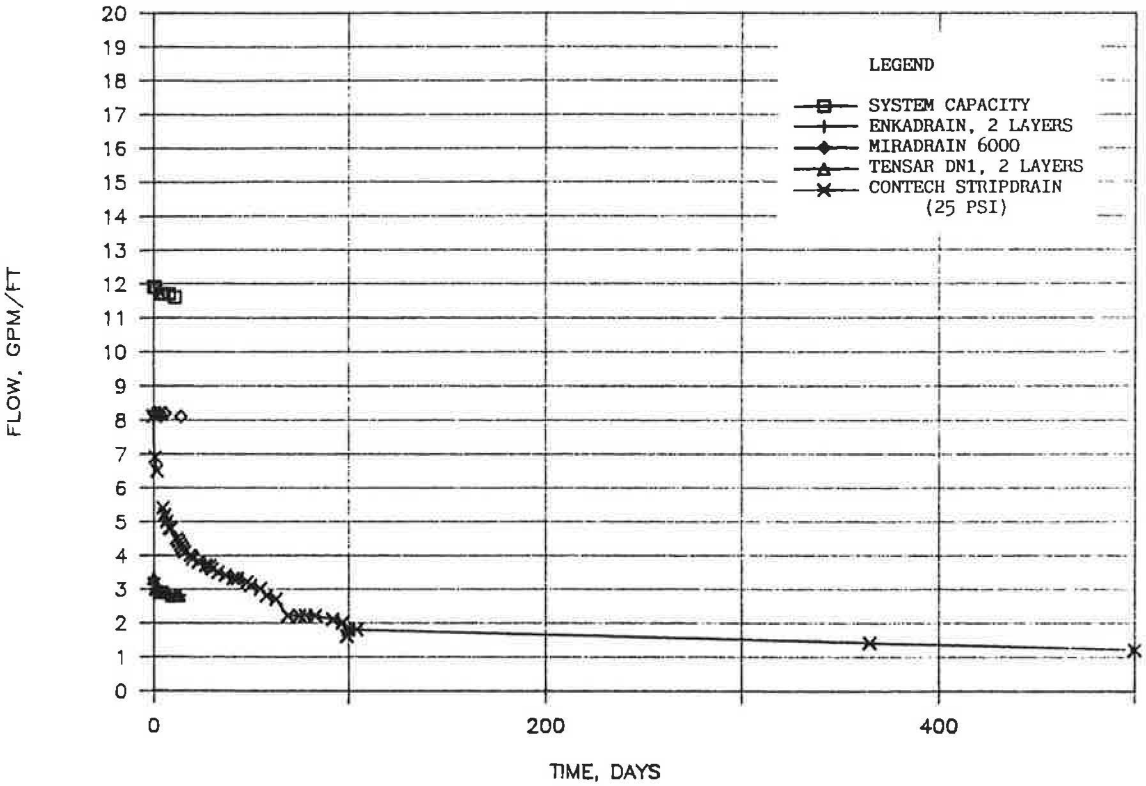

FIGURE 5 Equilibrium flow versus time (0–500 days, 30-psi confining pressure, 1.0 gradient): system capacity; Enkadrain, two layers; Miradrain 6000; Tensar DN1, two layers; Contech Stripdrain (25 psi).

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	Gradient: Equilibrium Flow, gpm/ft 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.

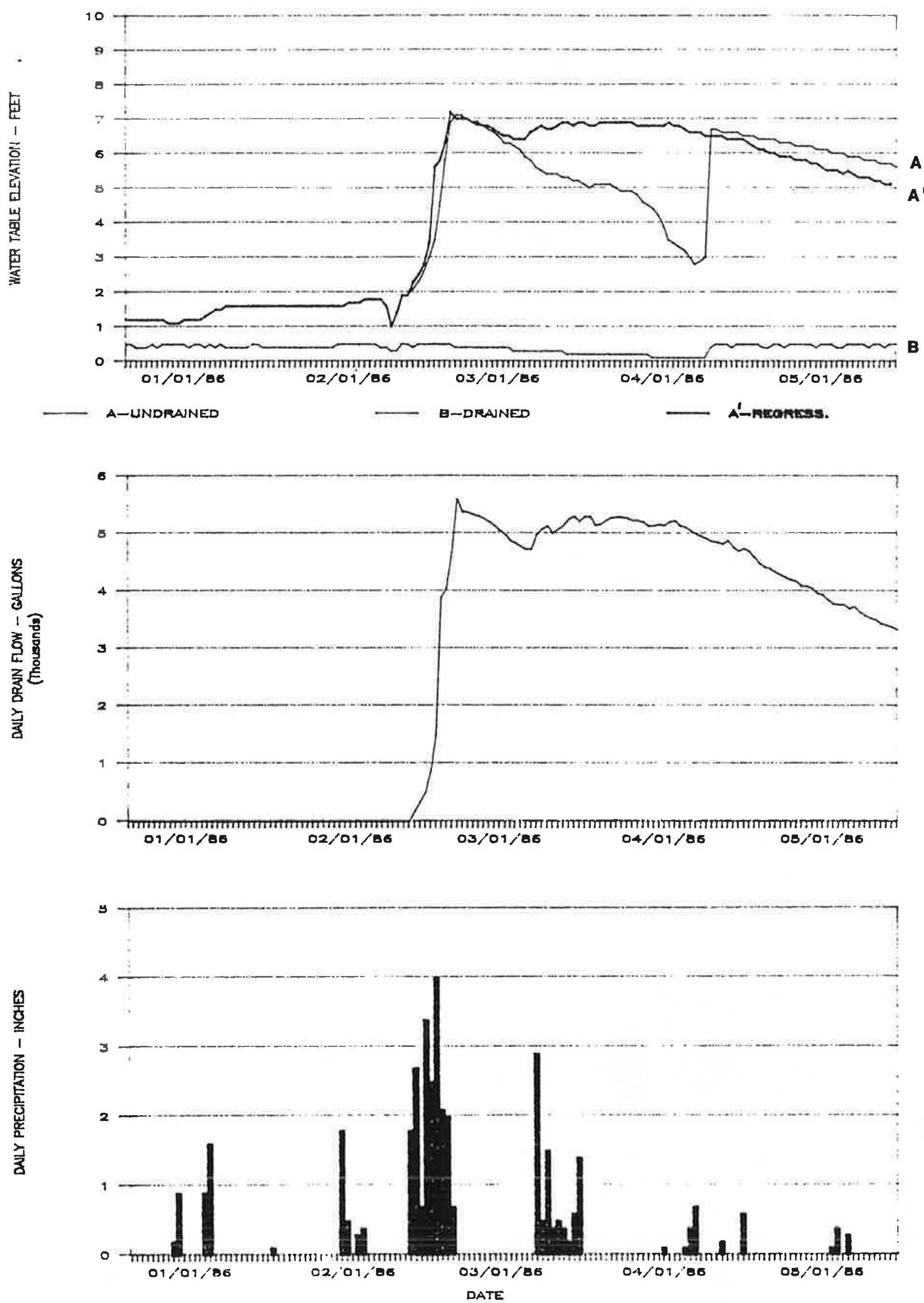

FIGURE 8 Site 17 on Stump Springs Road: (*top*) daily average water table elevation, (*middle*) daily drain flow, (*bottom*) daily precipitation.

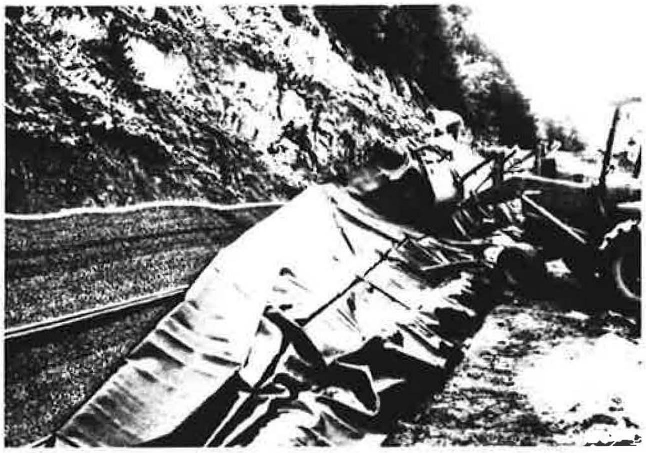

FIGURE 9 Installation of drain system on Mosquito Ridge Road.

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

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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

$$Q = kia \qquad (1)$$

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

$$V = \frac{1.486}{N} R^{\frac{2}{3}} S^{\frac{1}{2}} \qquad (2)$$

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

$$Q = VA \qquad (3)$$

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.

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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

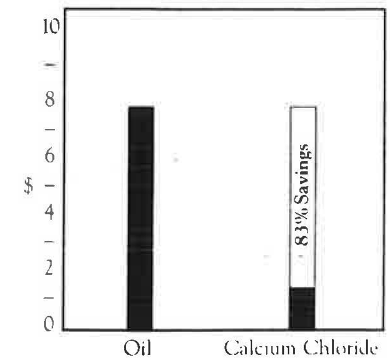


FIGURE 1 Cost comparison for unpaved road treatments.

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

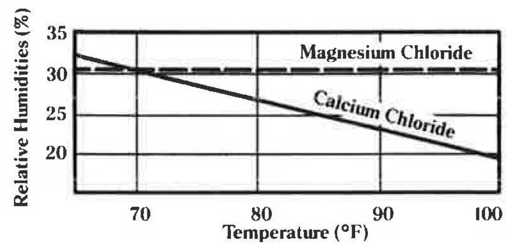


FIGURE 2 Temperature and humidity required to keep each chemical in solution.

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

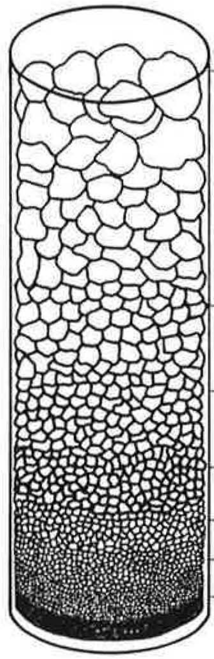
Compared with plain water, calcium chloride possesses a stronger moisture film due to its greater surface tension, reduced vapor pressure, and lower freezing point. These properties, in addition to the chemical's moisture-attraction capability and its ability to resist evaporation, provide seven benefits 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.



Sieve Designation	% Passing
1"	100
3/4"	70-100
3/8"	50-80
#4	35-65
#10	25-50
#40	15-30
#200	5-15

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.

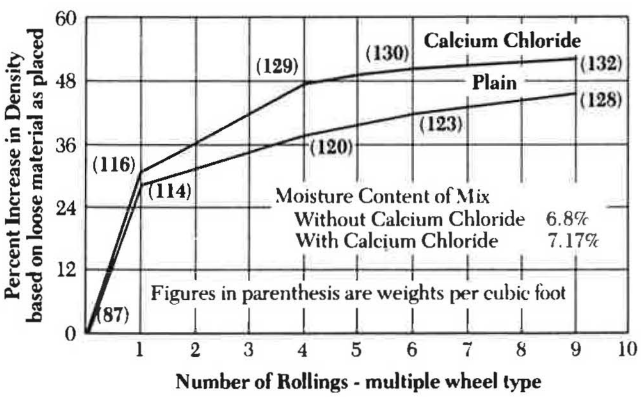


FIGURE 4 Compaction test results using calcium chloride versus plain water.

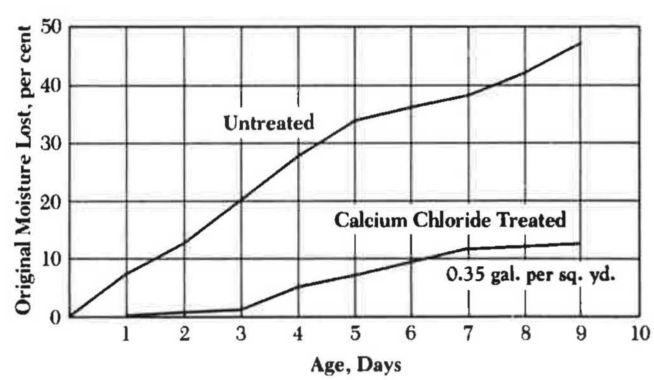


FIGURE 5 Moisture retention on road materials treated with calcium chloride versus untreated road materials.

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).

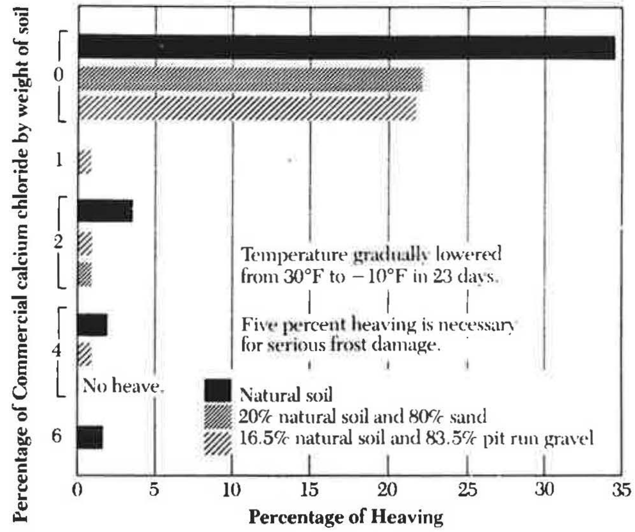


FIGURE 6 Effect of various amounts of calcium chloride on reduction of frost heaving in different soil mixtures.

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.

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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

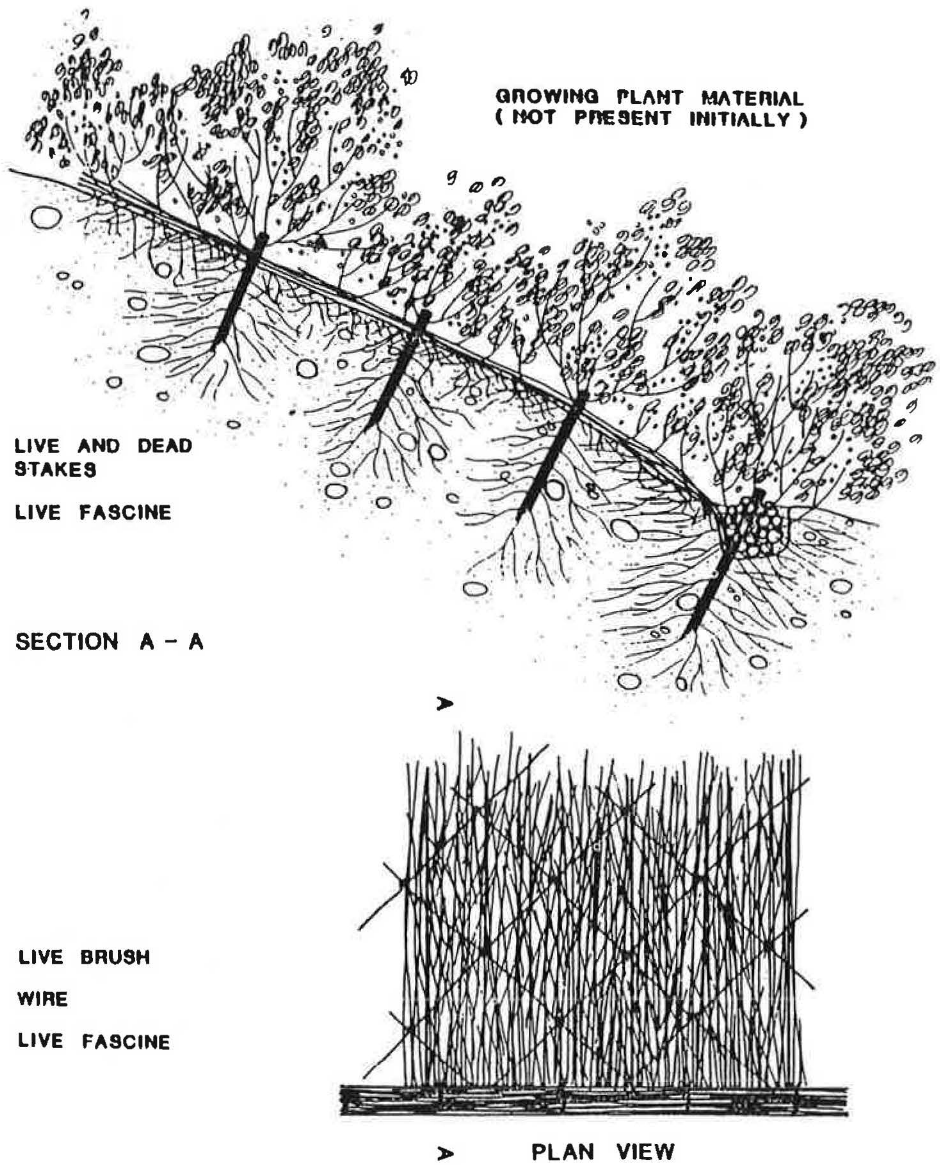

FIGURE 1 Brush mattress.

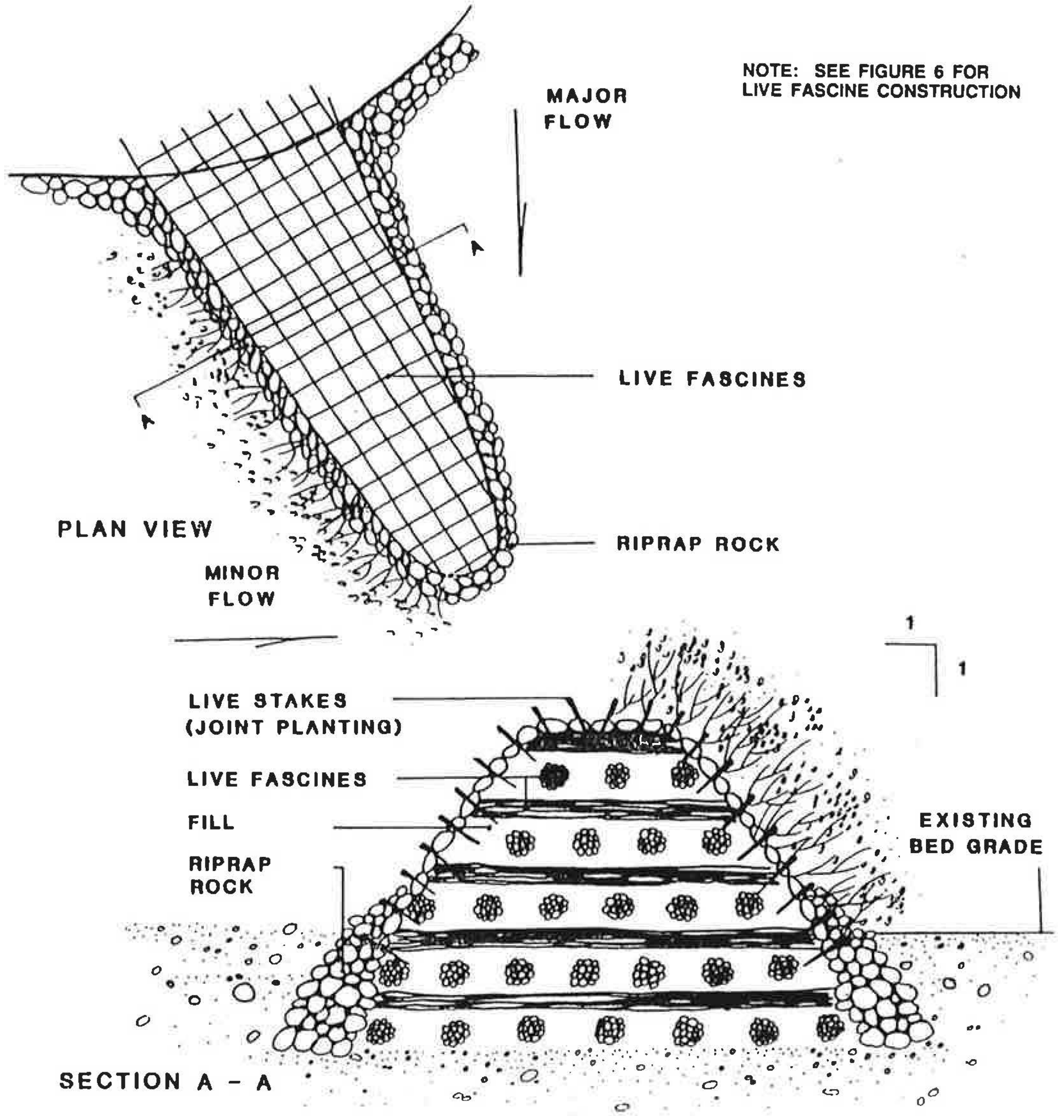

FIGURE 2 Live-boom construction.

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

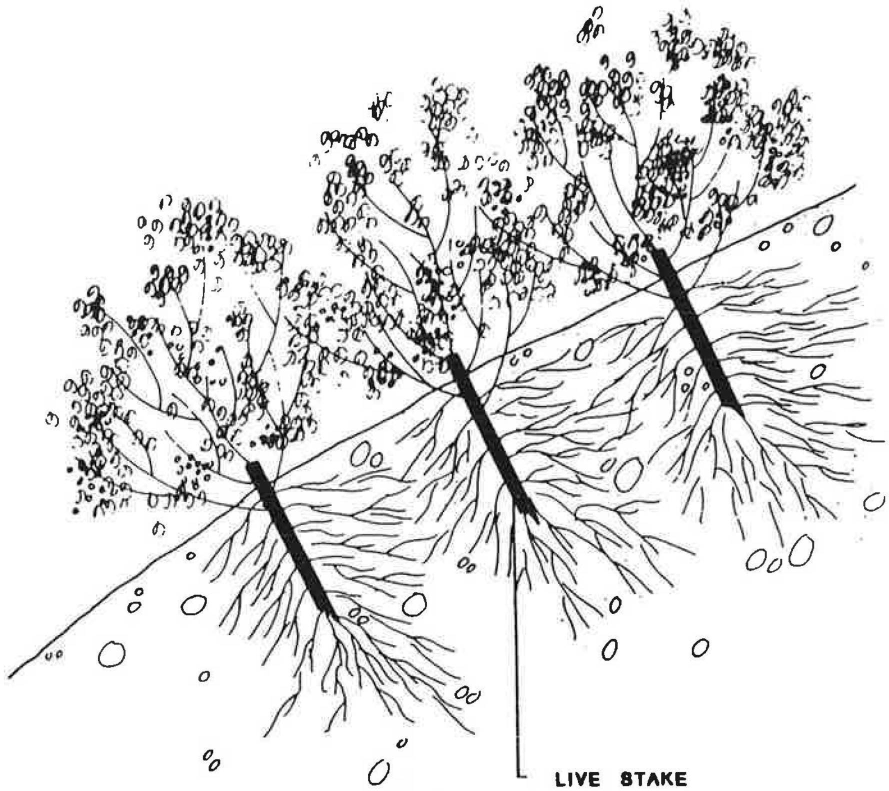

FIGURE 3 Live-stake construction.

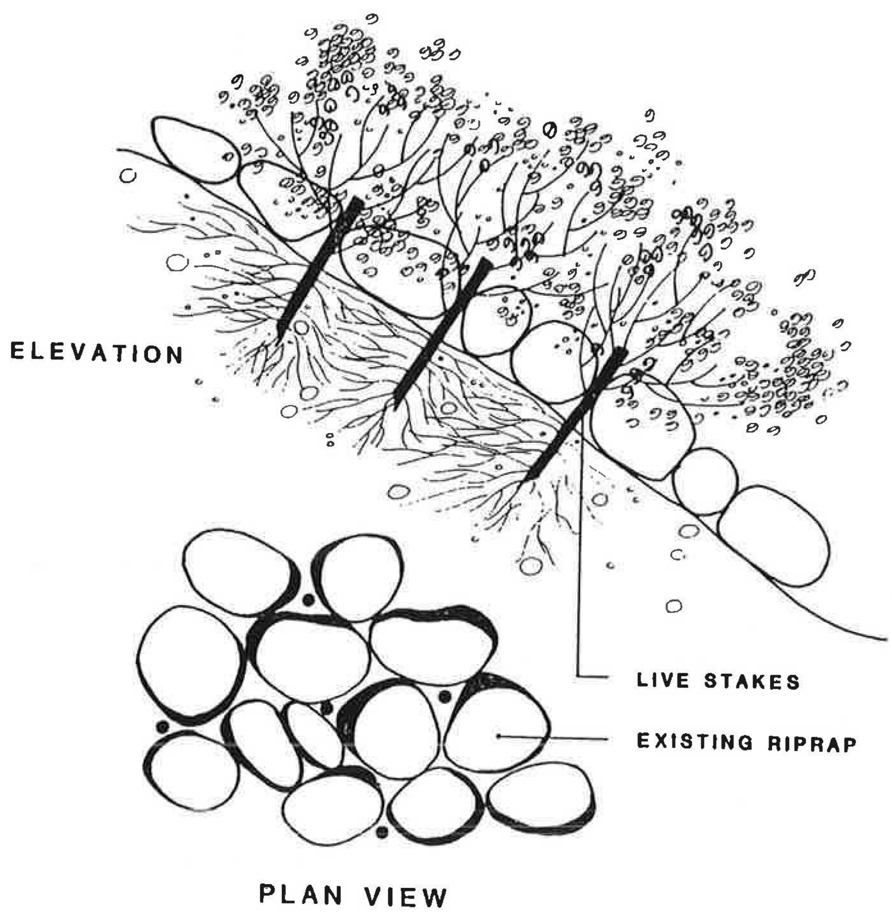

FIGURE 4 Joint planting.

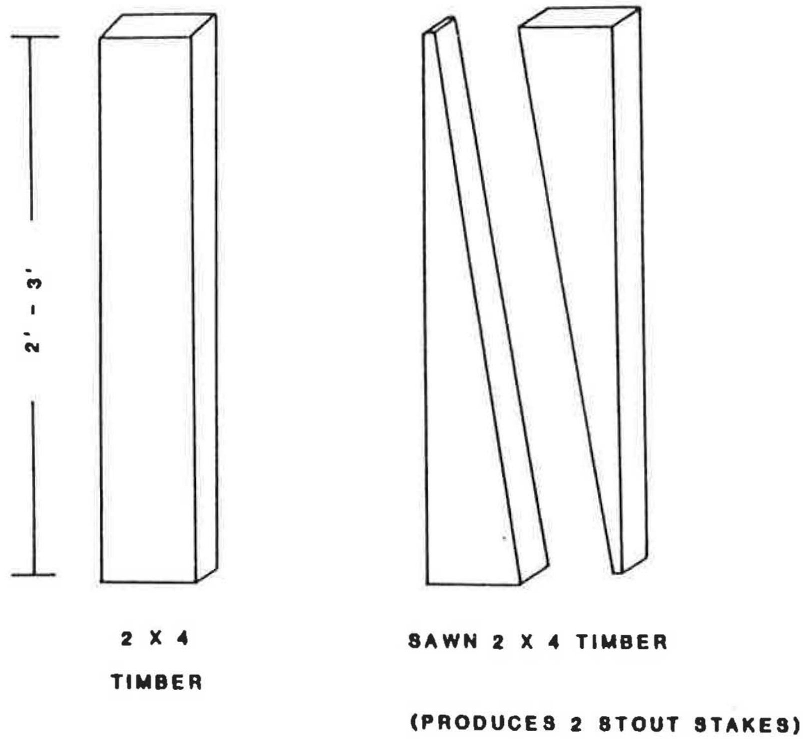

FIGURE 5 Stout stakes.

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

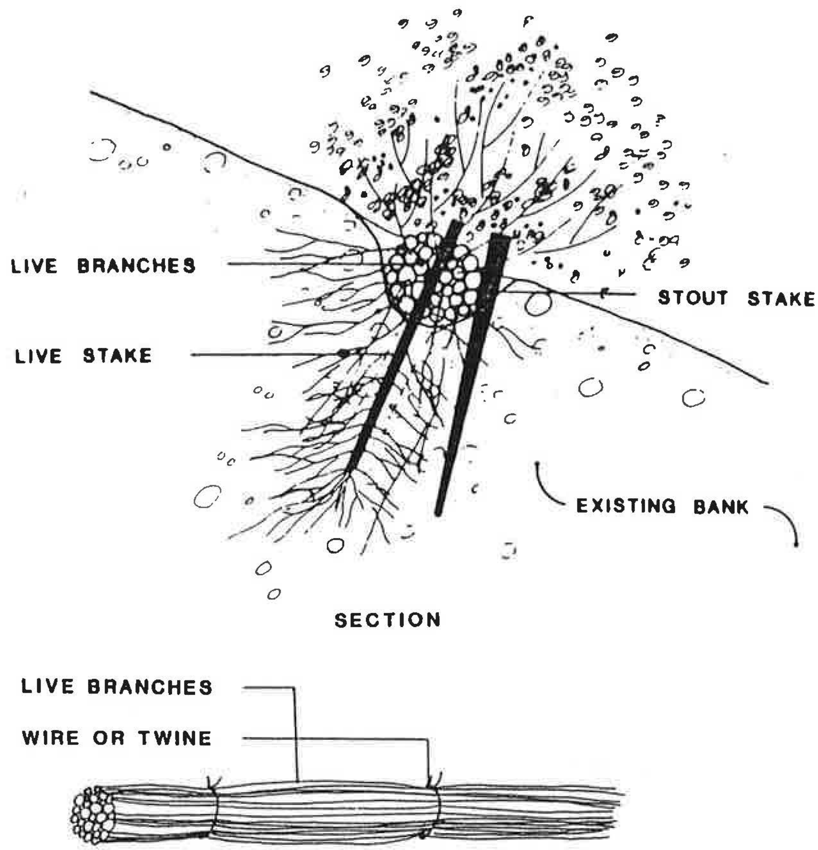

FIGURE 6 Live fascine bundle.

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 ≤ 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 alignment 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-

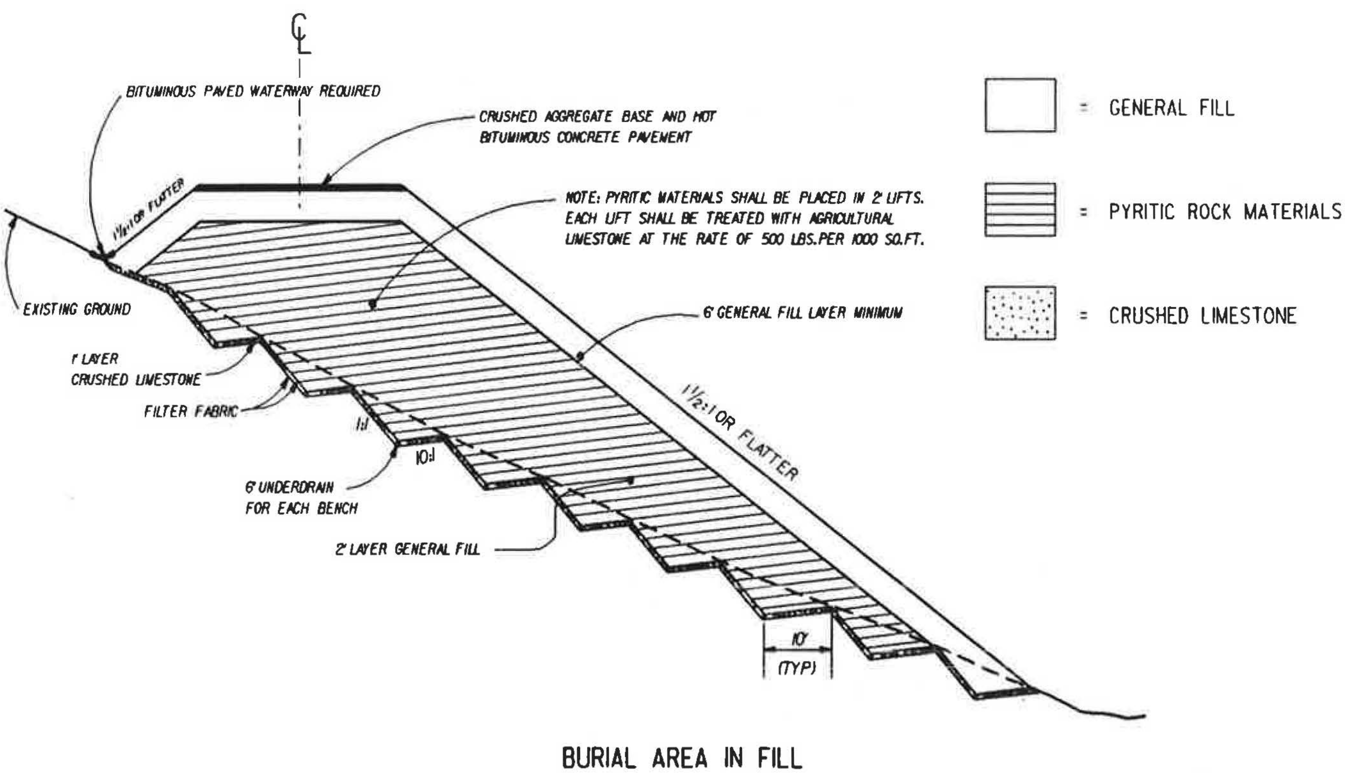

FIGURE 1 Method for treatment of acid-producing material.

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-

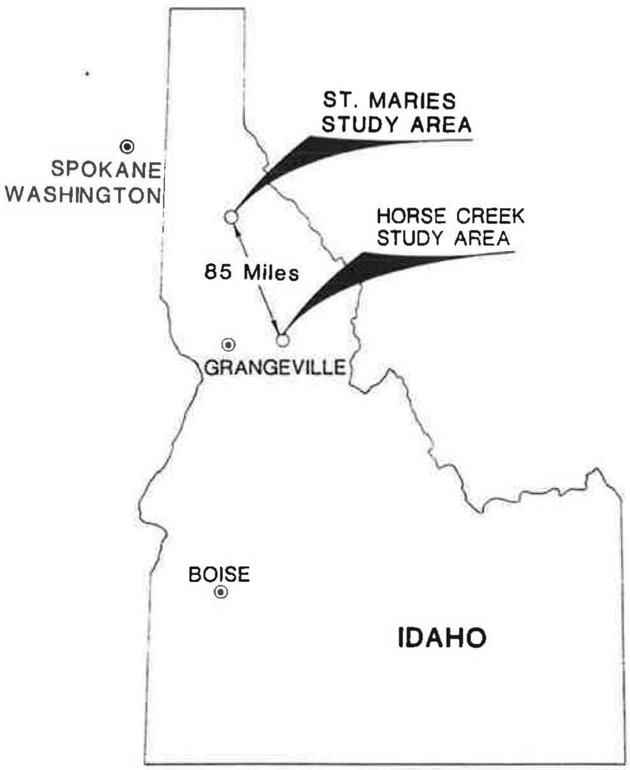

FIGURE 1 Project location.

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)

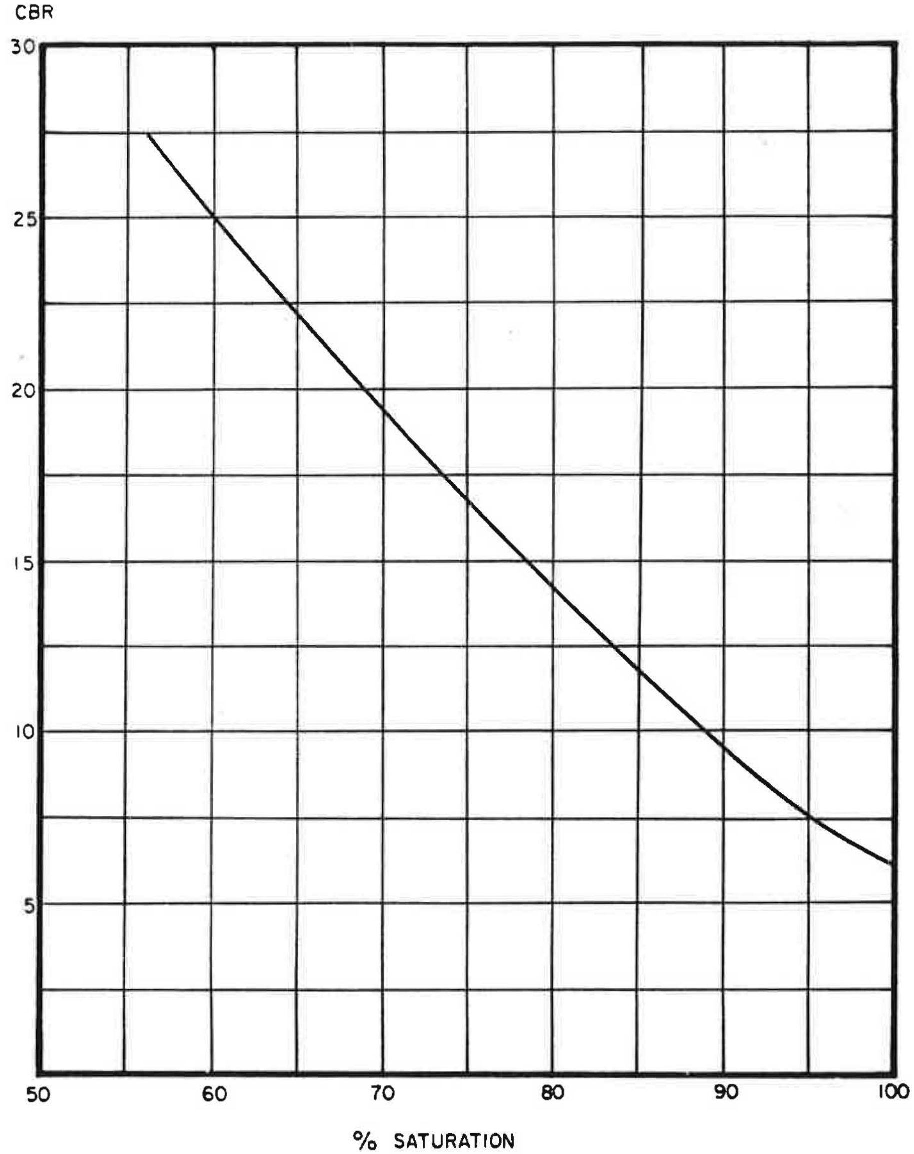

FIGURE 2 Example of a range of subgrade strength (CBR) values from a soil tested at varying moisture conditions.

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.

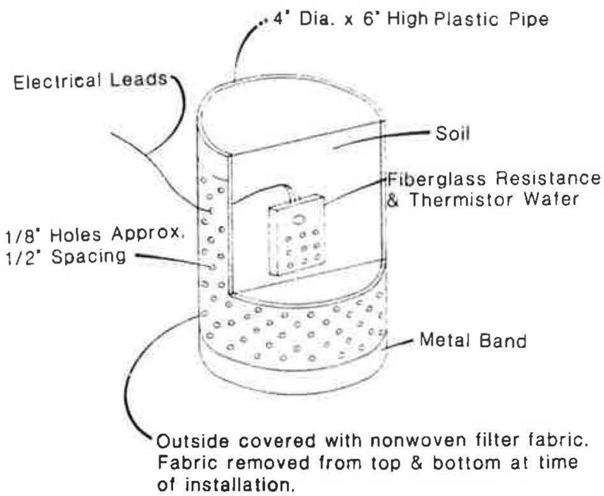

FIGURE 3 Electrical cell details.

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.

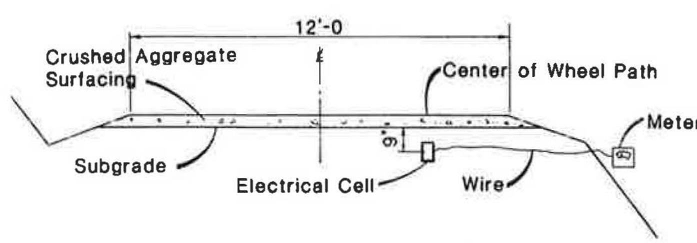

FIGURE 4 Electrical cell field installation.

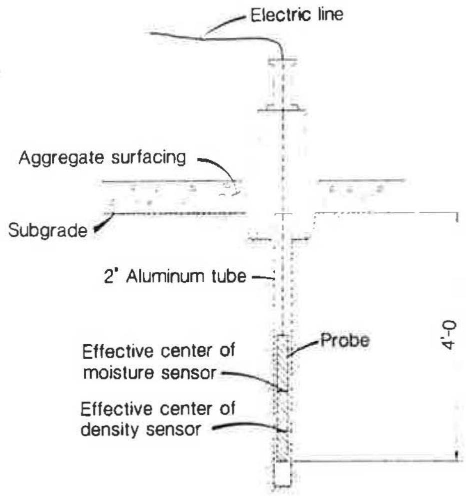

FIGURE 5 Nuclear depth probe schematic.

	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.

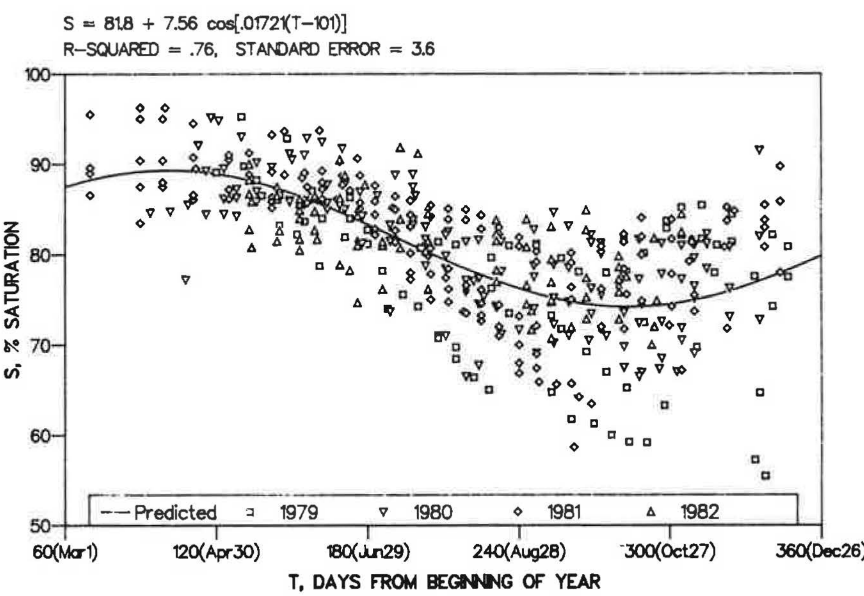

FIGURE 6 Total electrical cell data, cosine prediction.

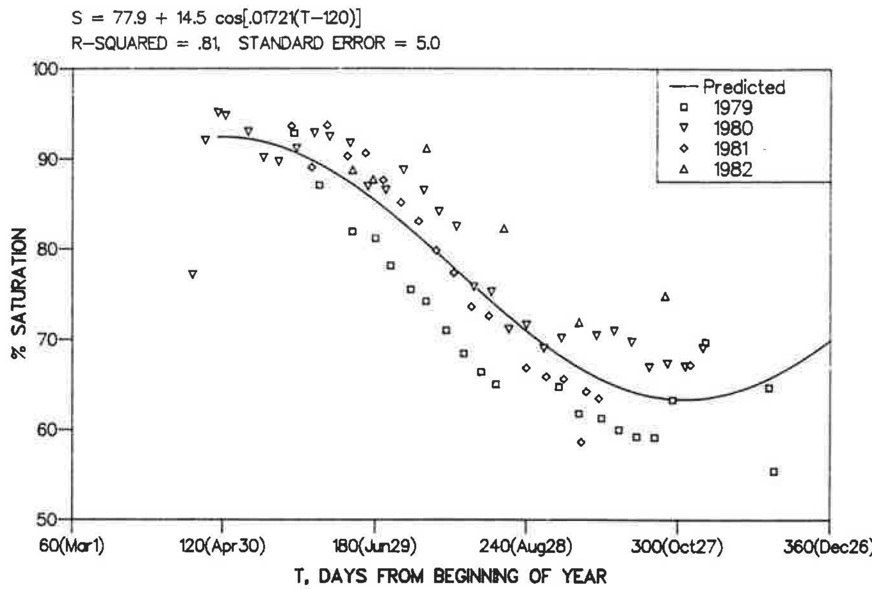

FIGURE 7 Horse Creek electrical cell data, cosine prediction.

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;

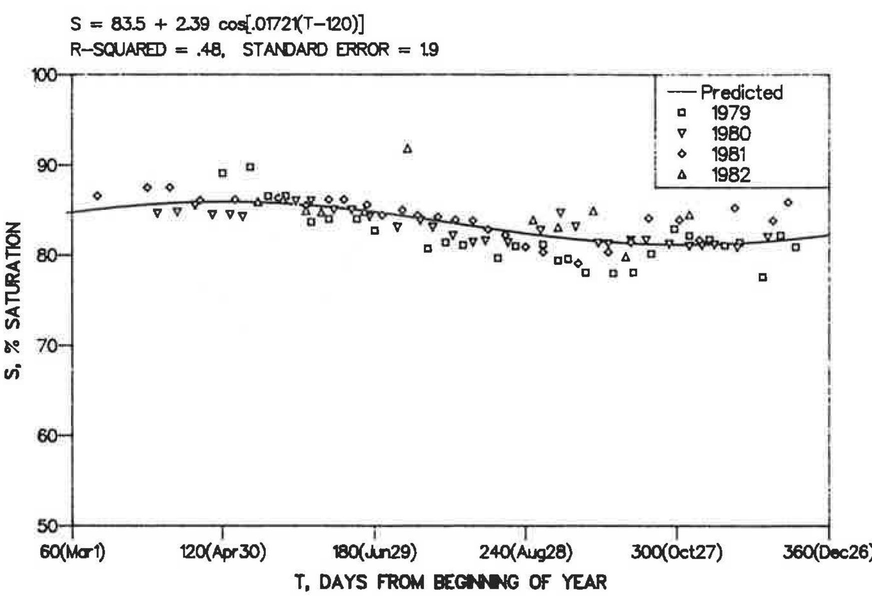

FIGURE 8 Staples electrical cell data, cosine prediction.

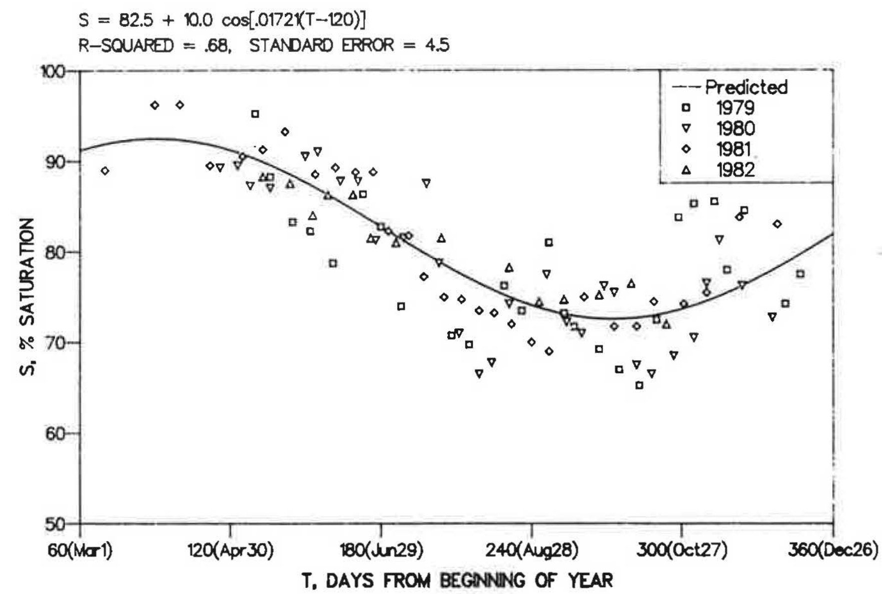

FIGURE 9 Xmas electrical cell data, cosine prediction.

- 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.

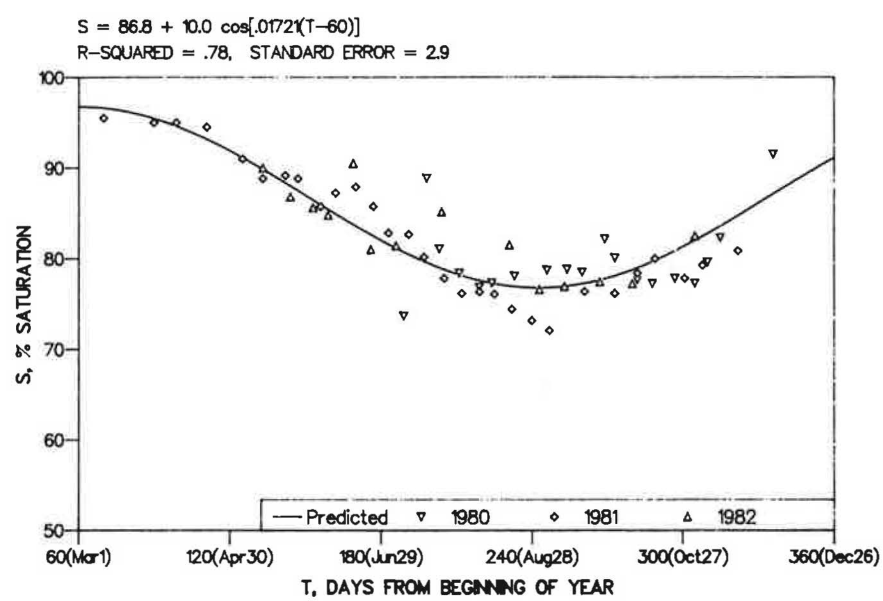

FIGURE 10 Merry electrical cell data, cosine prediction.

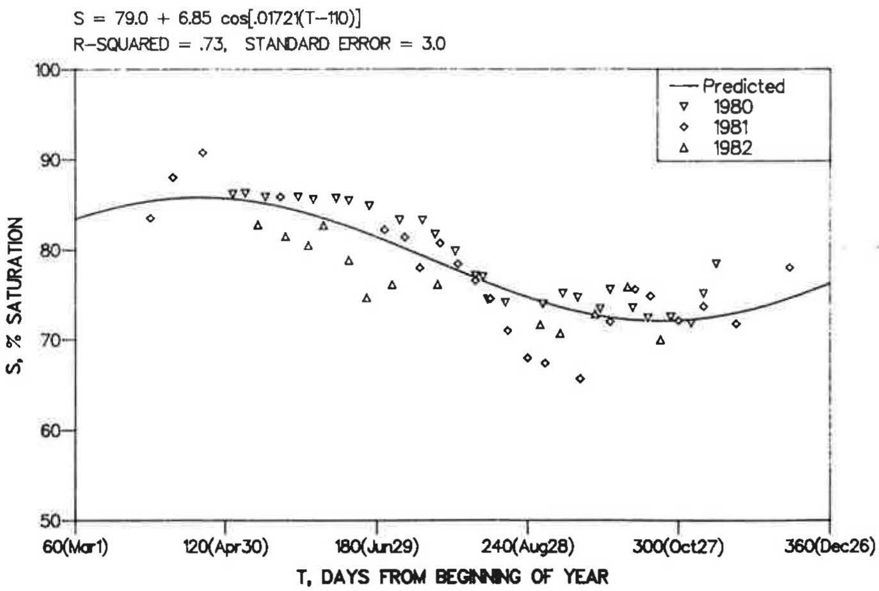

FIGURE 11 Gold electrical cell data, cosine prediction.

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.

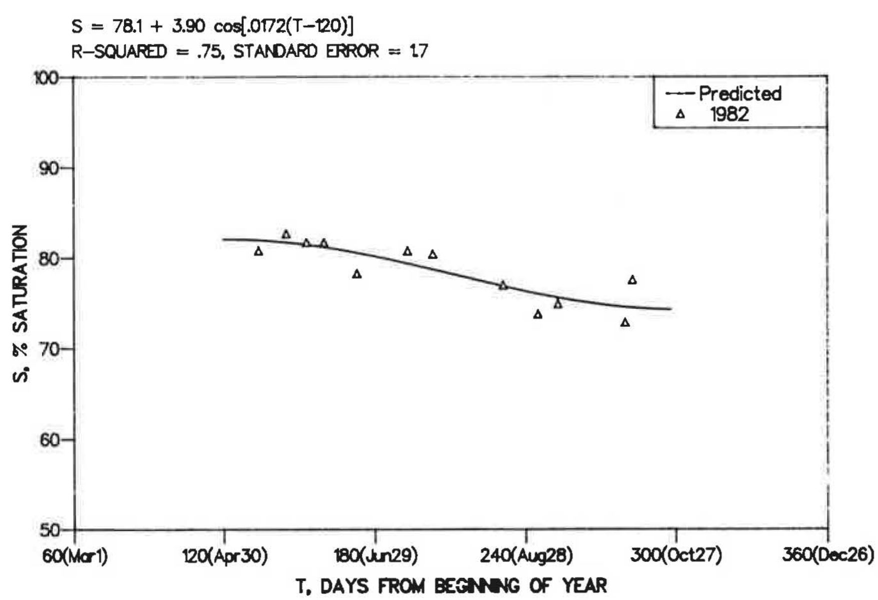

FIGURE 12 Emerald electrical cell data, cosine prediction.

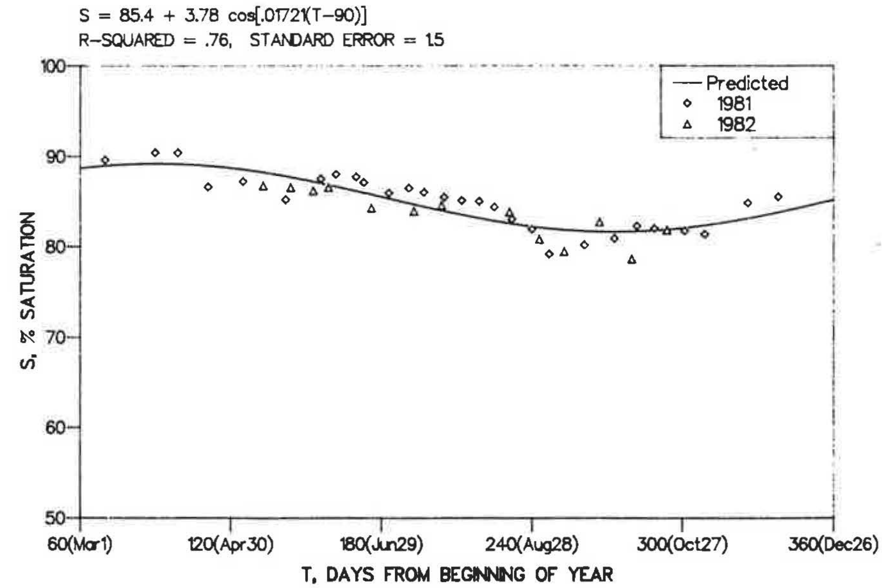

FIGURE 13 Elk electrical cell data, cosine prediction.

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

$$API_n = K(API_{n-1}) + P_n \tag{3}$$

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.

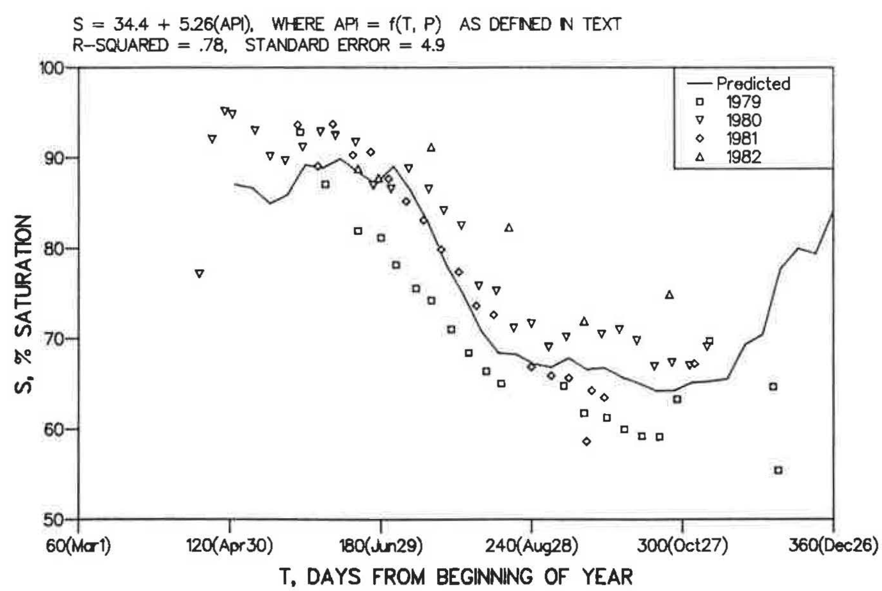

FIGURE 14 Horse Creek electrical cell data, API prediction.

TABLE 2 NUCLEAR DEPTH PROBE MOISTURE CONTENTS AND DRY DENSITIES, HORSE CREEK, 1980

	DATE					
Depth below Subgrade, ft.	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

$$S = 34.4 + 5.26(\text{API}) \tag{4}$$

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

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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.

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-

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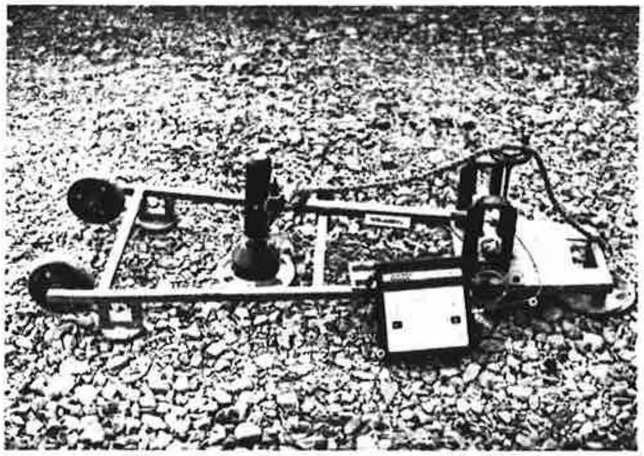

FIGURE 1 First ODIN prototype with data processor.

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.

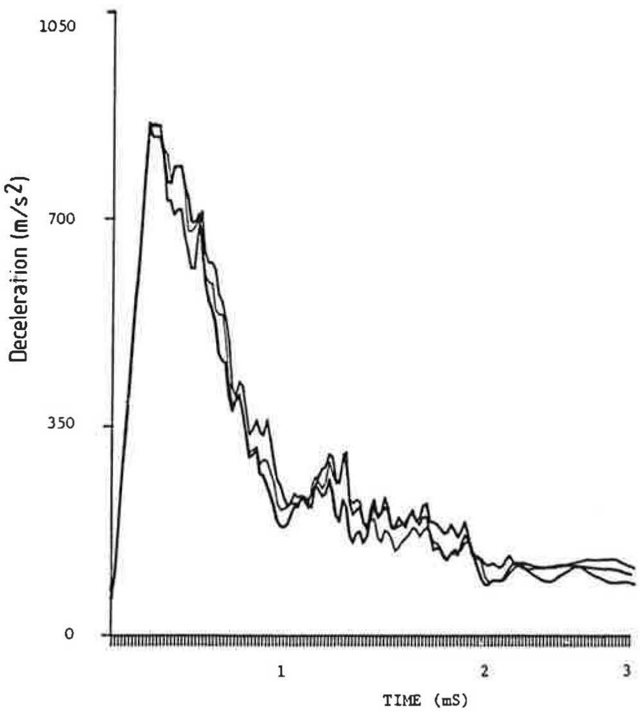

FIGURE 2 Accelerometer output for three successive tests demonstrating repeatability.

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.

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).

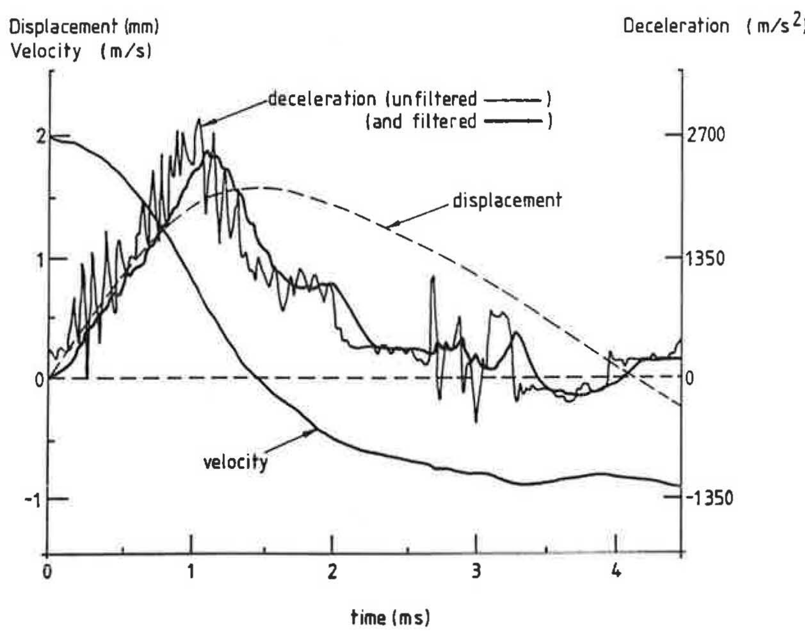

FIGURE 3 Typical unfiltered, filtered, and integrated accelerometer signal on impact.

$$E_r = k_1\theta^{k_2}$$

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.

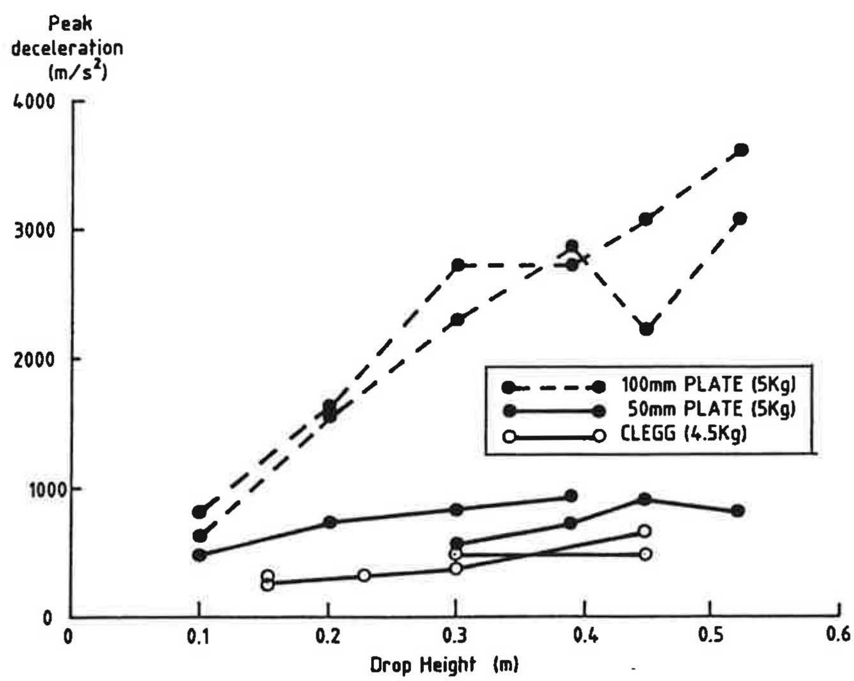

FIGURE 4 Relationship between peak deceleration and drop height for varying impact plate sizes.

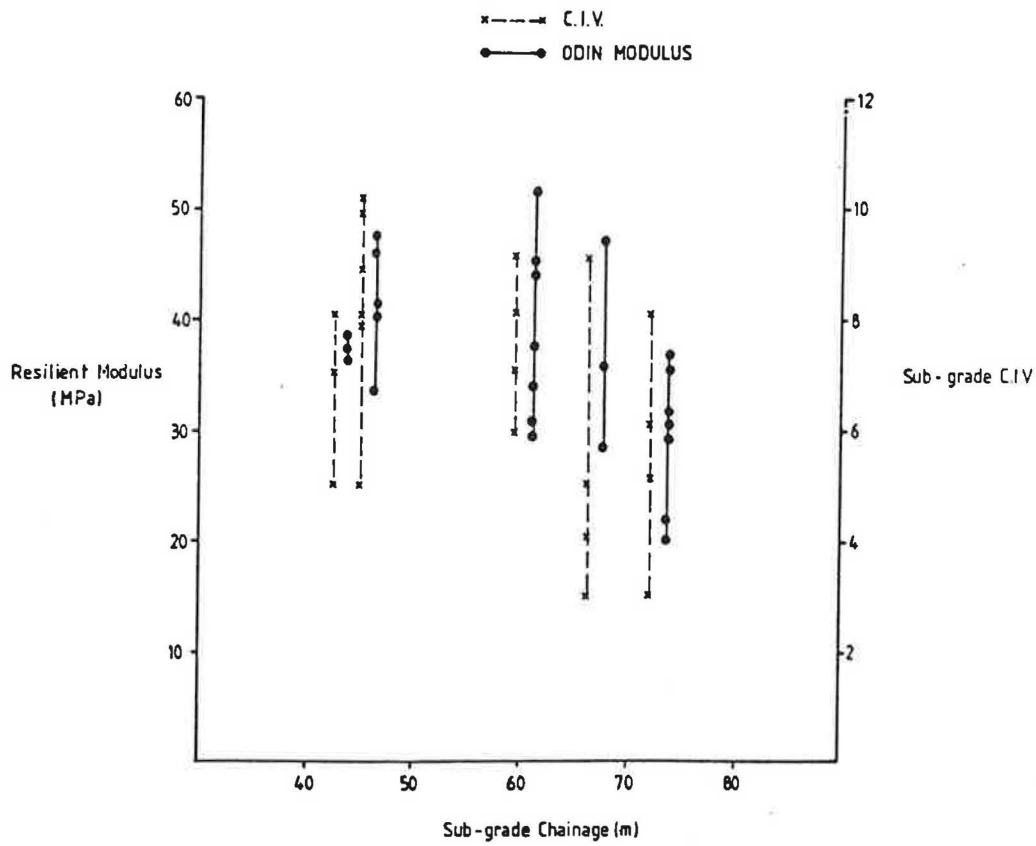

FIGURE 5 Variation of resilient modulus and CIV with increased chainage on clay subgrade.

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

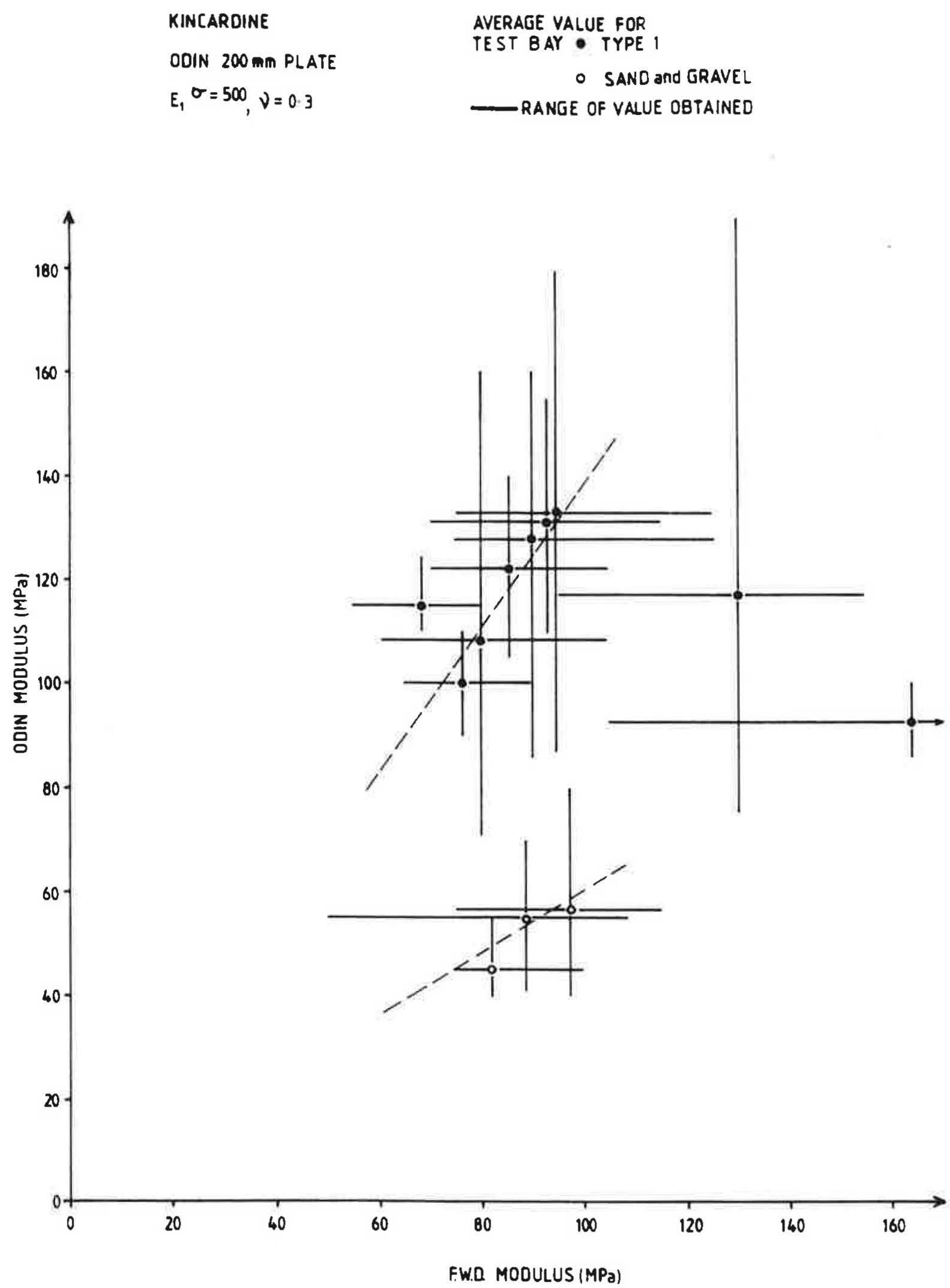

FIGURE 6 Resilient modulus and FWD back-calculated modulus for 12 subbase constructions.

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.

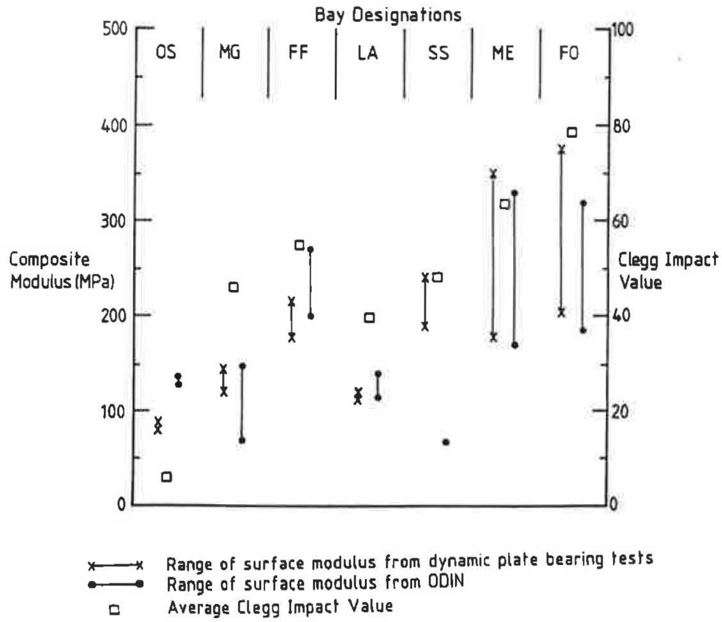

FIGURE 7 Composite modulus (ODIN and FWD) and CIV for seven construction bays.

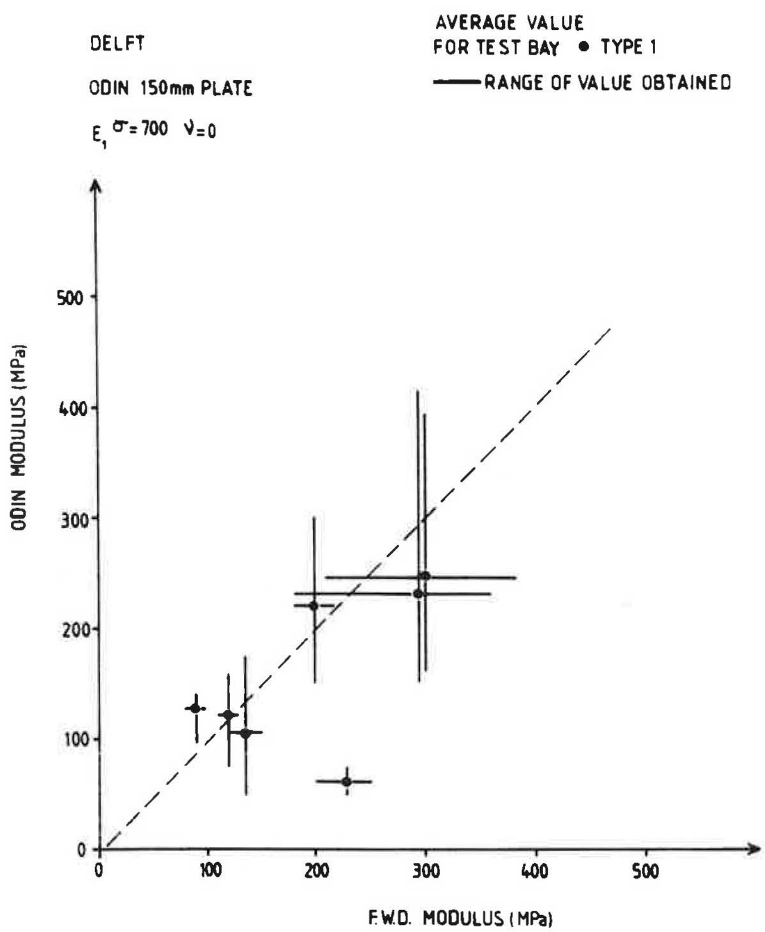

FIGURE 8 FWD and ODIN modulus values for bays tested.

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		Rock Type	
Chainage	Degree	Chainage	Type
7.0-7.8	36-45°	7.0-7.8	Fractured slate
7.8-8.0	45-50°	7.8-8.2	Colluvium
8.0-9.1	36-45°	8.2-8.6	Slate
9.1-9.2	46-50°	8.6-8.8	Colluvium
9.2-9.4	>51°	8.9-9.0	Interbedded quartzite + claystone
9.4-10.0	46-50°	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		**Landslides**	
Chainage	Type	Chainage	Type
7.0-9.0	Sparse brushes	7.1-7.2	Medium slump
		7.2-7.3	Medium planar rockslide
9.0-10.0	Sparse trees	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		**Geologic Structure**	
Chainage	Type	Chainage	Structure
7.0-9.0	Dendritic	7.0-8.0	Crushed and fractured rock
9.0-10.0	Trellis	7.8-9.9	Folded area
		8.2-8.5	Folded area
		9.2-9.4	Crushed rock
Landuse		**Groundwater**	
Chainage	Type	Chainage	Condition
7.0-10.0	Non - cultivated	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.

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 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

TABLE 5 EXPLANATION OF HAZARD DIAGNOSIS

Location: 7.0 to 7.3 km Type of failure: Planar rock slide Extent of failure: Small	Location: 7.3 to 7.7 km Type of failure: Wedge failure 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

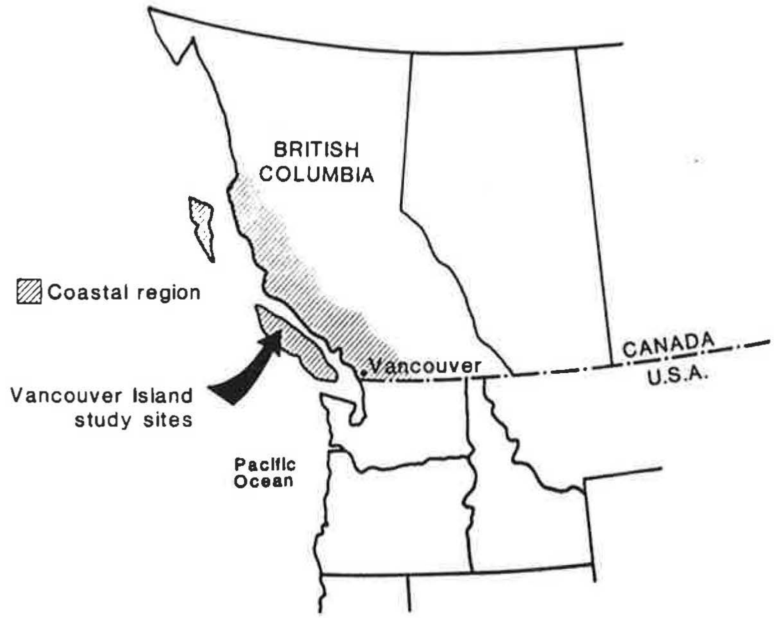

FIGURE 1 FERIC's rock drill study sites.

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

FIGURE 2 Tamrock Logmatic at Kelsey Bay.

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.

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-

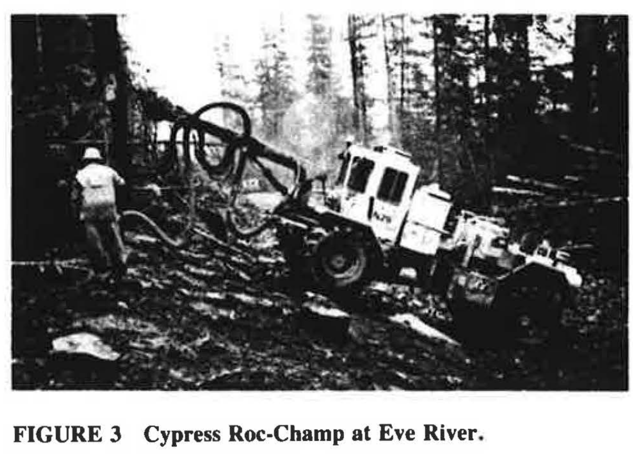

FIGURE 3 Cypress Roc-Champ at Eve River.

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.

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 5 Finning M32FA at Cameron.

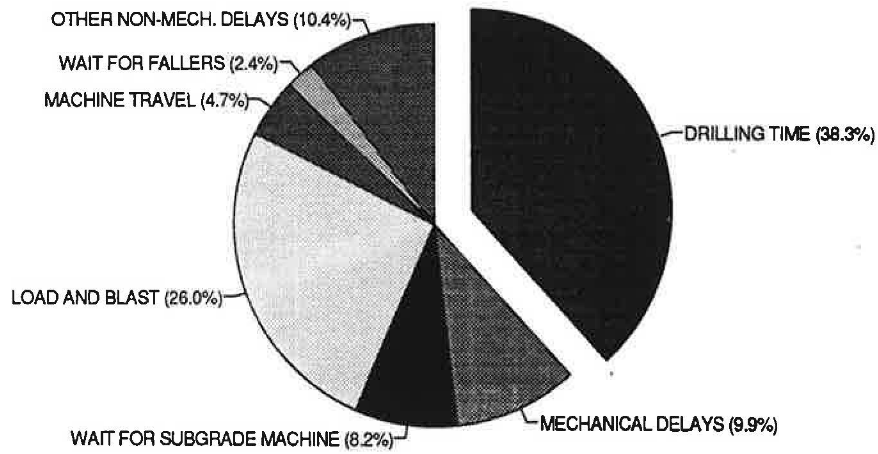

FIGURE 6 **Time distribution for the Kelsey Bay study.**

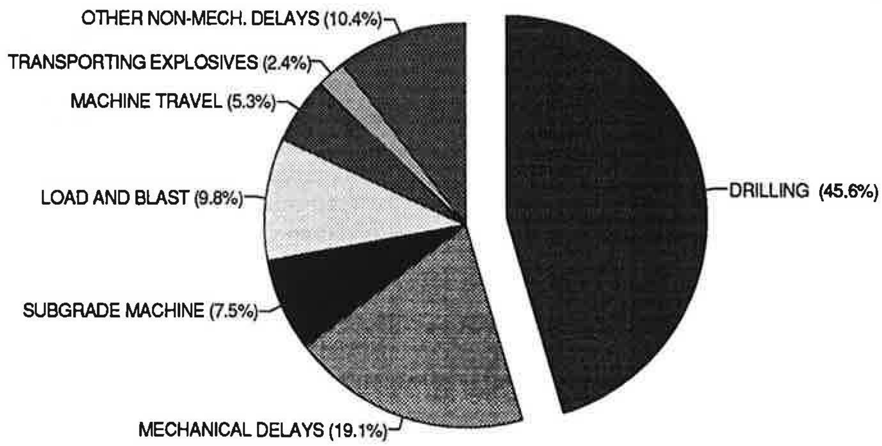

FIGURE 7 **Time distribution for the Eve River study.**

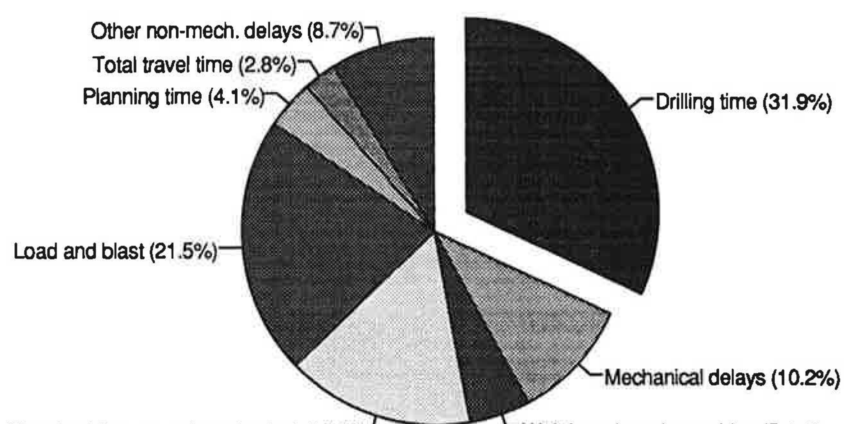

FIGURE 8 **Time distribution for the Englewood study.**

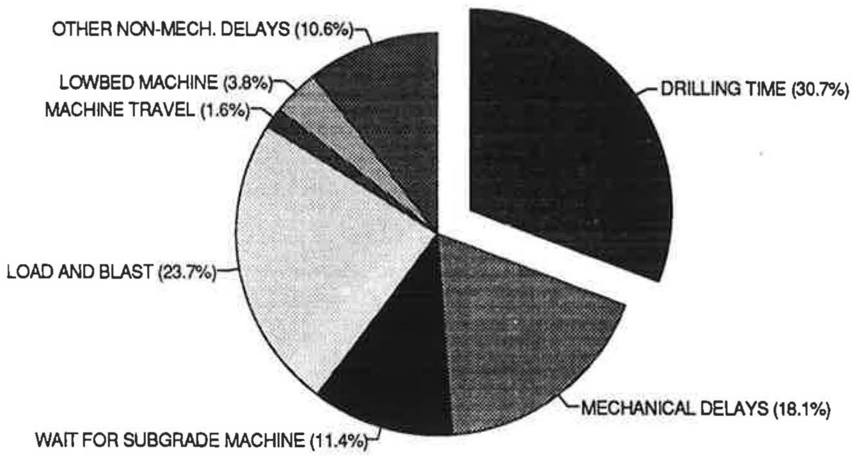

FIGURE 9 Time distribution for the Cameron study.

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 (Total no. moves • 100) / 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

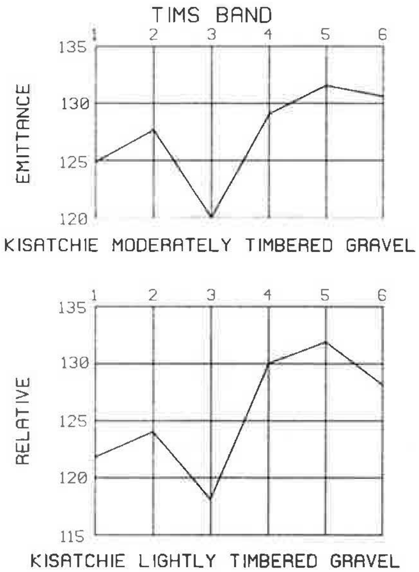

FIGURE 2 TIMS six-band spectral signatures for gravel deposits on the Kisatchie National Forest near Alexandria, La.

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

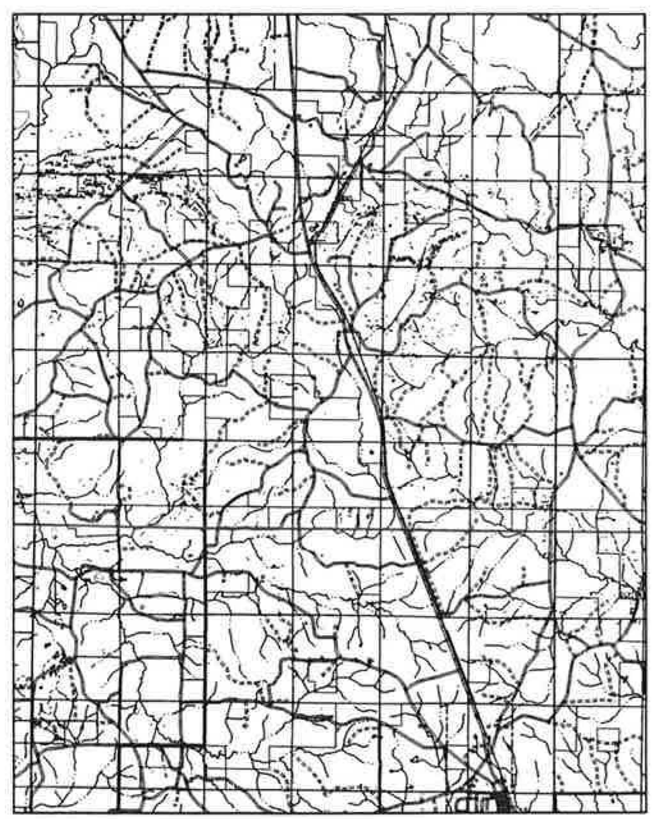

FIGURE 3 A gravel inventory map of the Williana, La., area. Hatched areas, indicating gravel deposits, were generated by processing rectified TIMS imagery and converting to ARC–INFO for plotting with an electrostatic plotter.

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 environmental 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.

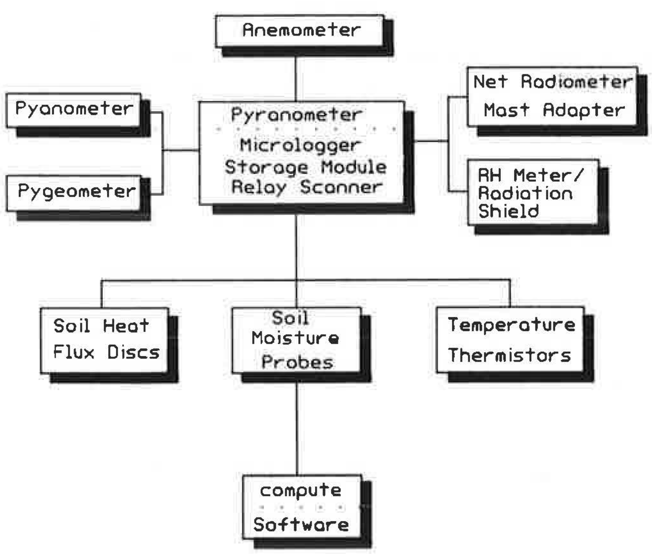

FIGURE 4 A block diagram of NASA's micrometeorological station installed on the Desoto National Forest.

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 investigations 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

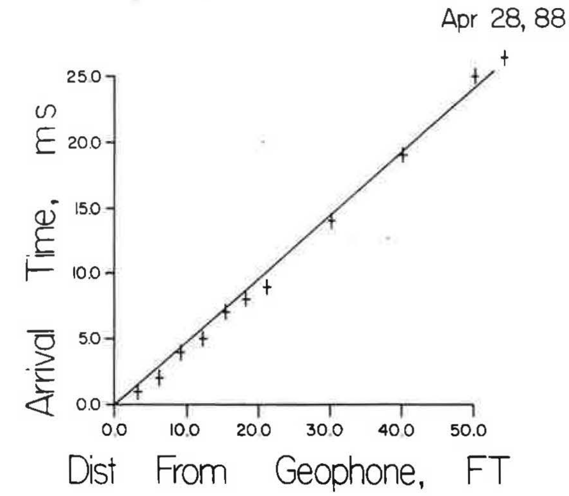

FIGURE 2 Typical arrival time versus distance for unfrozen subgrade soil.

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*):

$$T_1 = 0.5 t_i V_1 V_2 / (V_2^2 - V_1^2)^{1/2} \quad (1)$$

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.

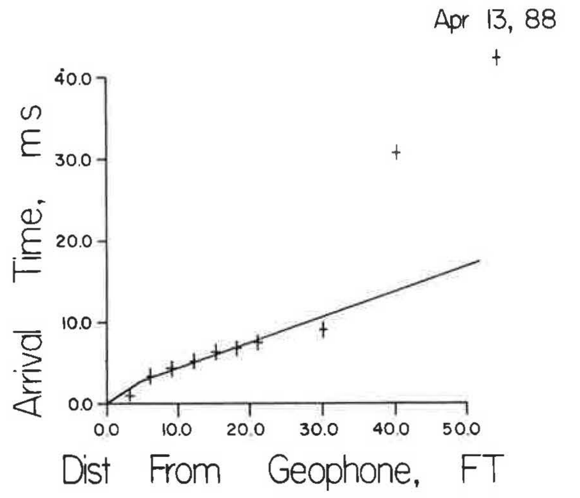

FIGURE 3 Typical arrival time versus distance for partially thawed subgrade soil.

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,

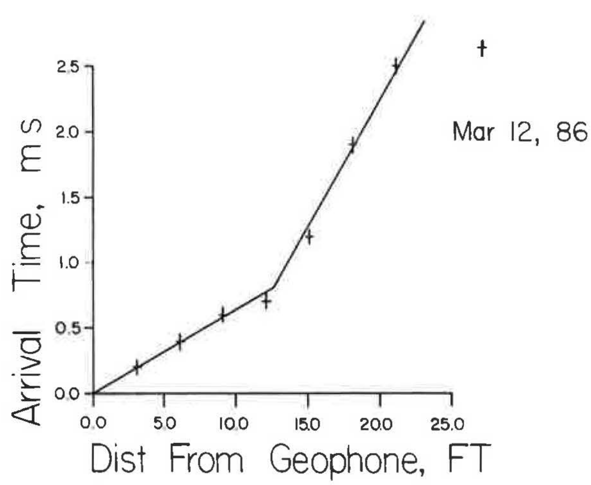

FIGURE 4 Typical arrival time versus distance for frozen subgrade soil.

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:

$$V = 2784.0 - 152.4T + 3.0T^2 \qquad (2)$$

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:

$$E = V^2p(1 + \mu)(1 - 2\mu)/(1 - \mu) \qquad (3)$$

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

$$CIV = 29.0 - 0.48T + 0.020T^2 \tag{5}$$

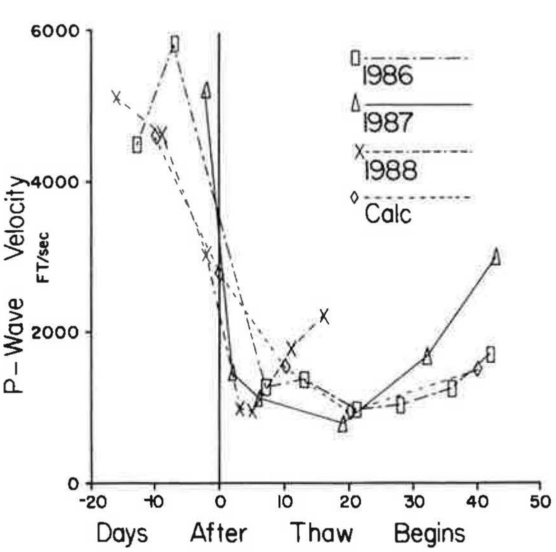

FIGURE 5 Summary P-wave velocity versus days after thaw begins.

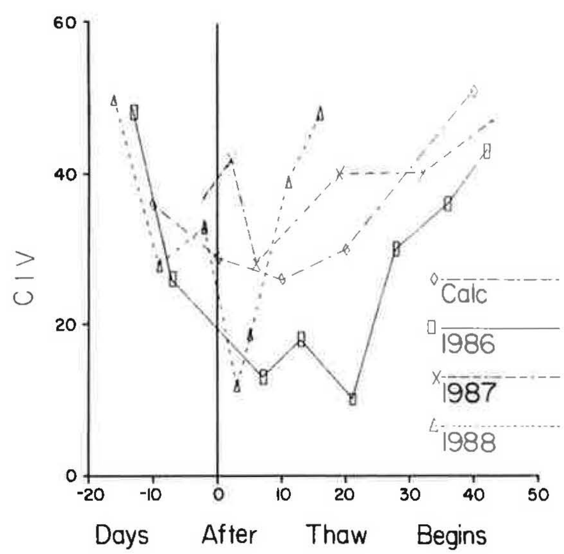

FIGURE 6 Summary CIV versus days after thaw begins.

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

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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.

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.

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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.

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).

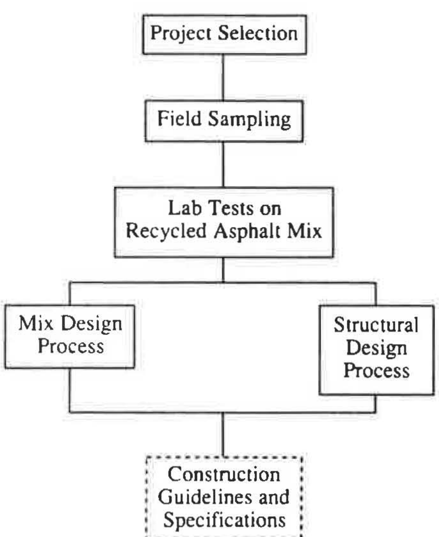


FIGURE 1 Preconstruction steps for CIR.

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

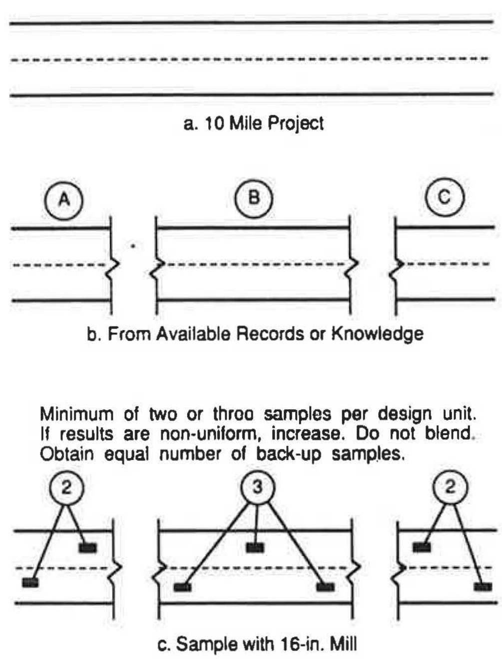


FIGURE 2 Suggested field sampling.

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:

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,

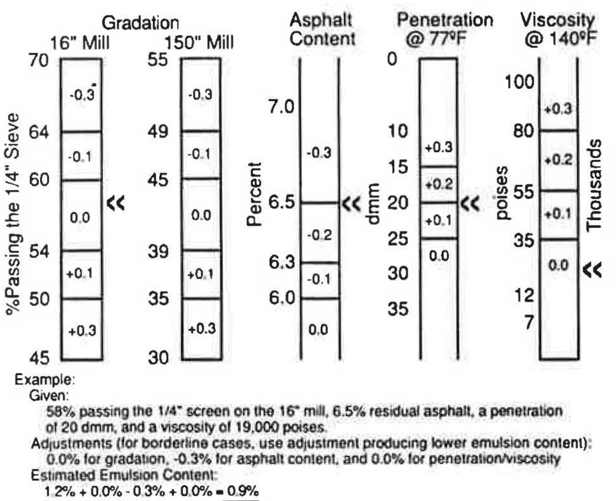


FIGURE 3 Emulsion content adjustments for gradation, asphalt content, and asphalt softness.

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

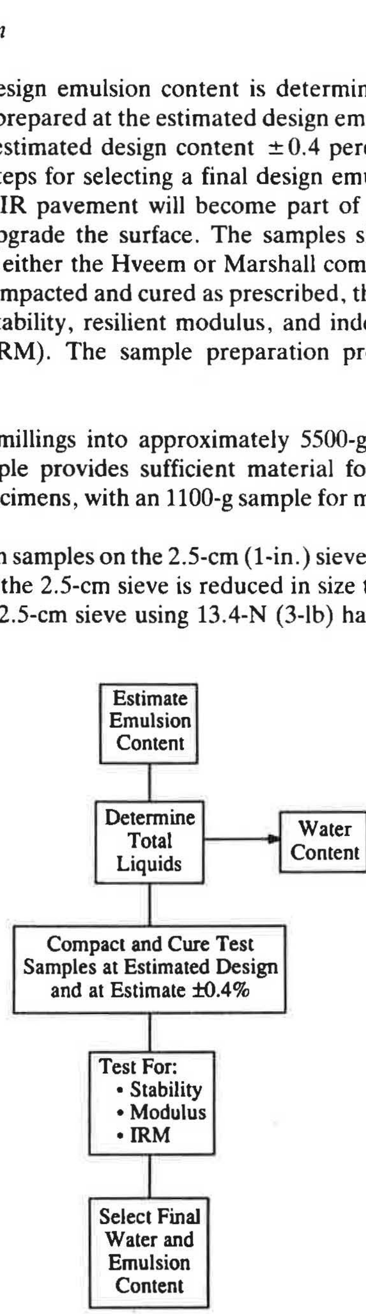


FIGURE 4 Suggested mix design process—future projects.

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	0-11	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	0-9?	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.)		Thermal Crack Spacing (ft)	Flushing	Fatigue Cracks	Maint. Work	Rating	Notes
									Lt.	Rt.						
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	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-150S 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

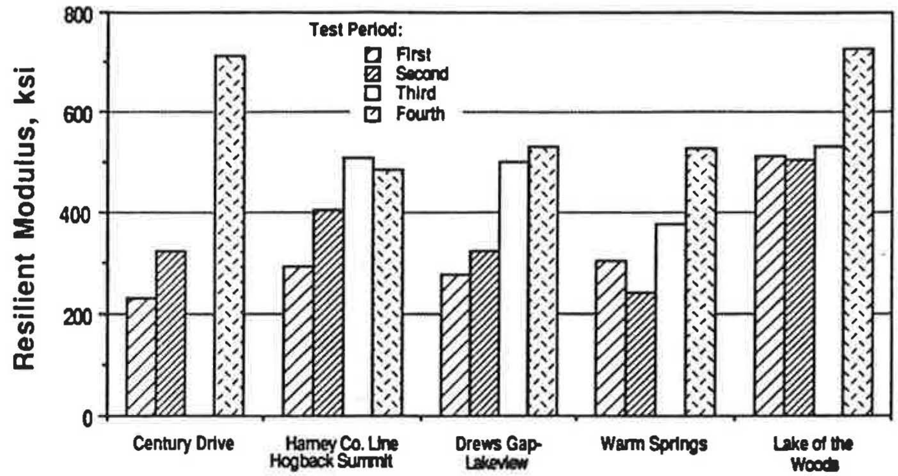
Typical Costs

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



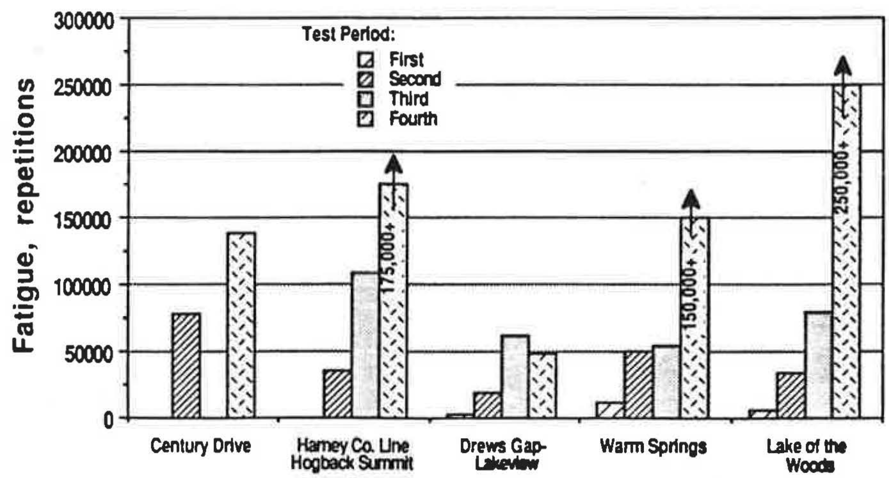


FIGURE 5 Resilient modulus (*top*) and fatigue (*bottom*) test results.

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.

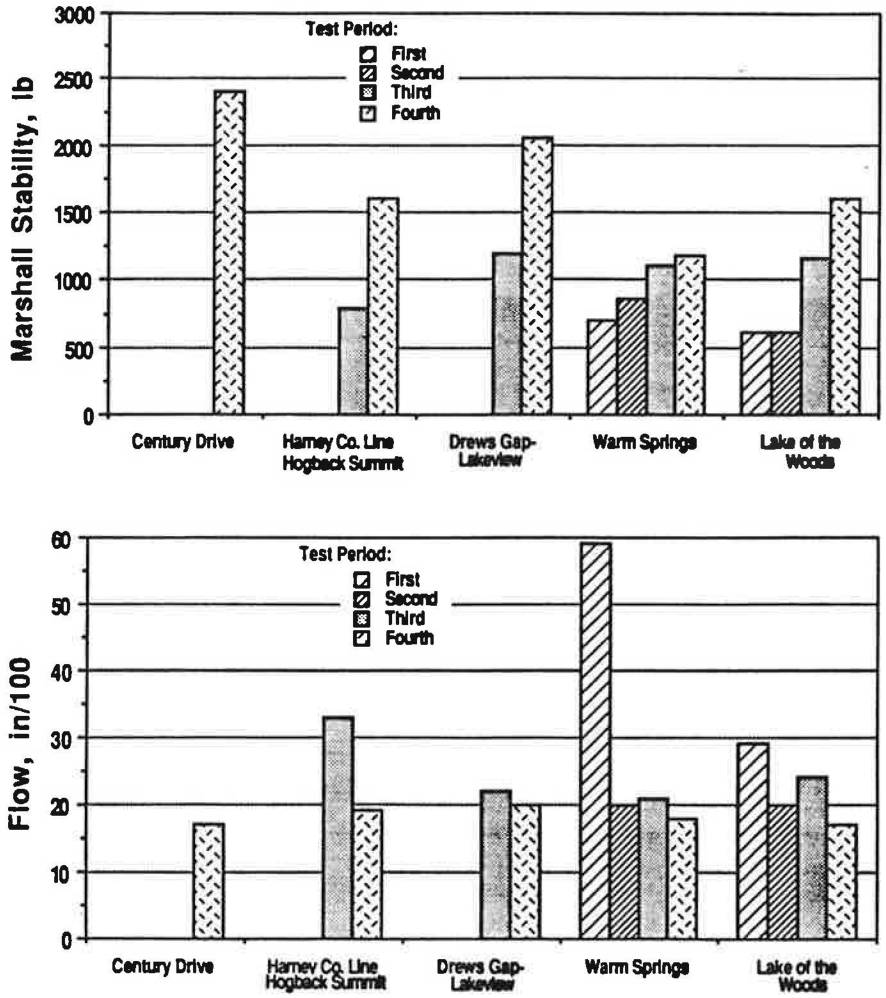


FIGURE 6 **Marshall stability (*top*) and flow (*bottom*) test results.**

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

PRITHVI S. KANDHAL

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] in.	mm	Volume of Specimen, cm^3	Correlation Ratio
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).

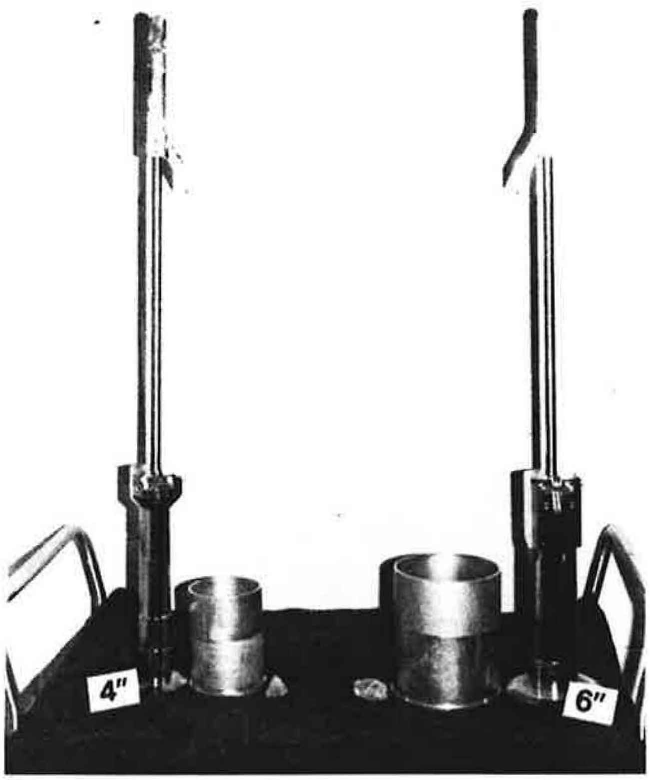

FIGURE 1 Mold and hammer assembly for 4- and 6-in.-diameter specimens. Aggregate particles of 1- and 2-in. maximum size are also shown.

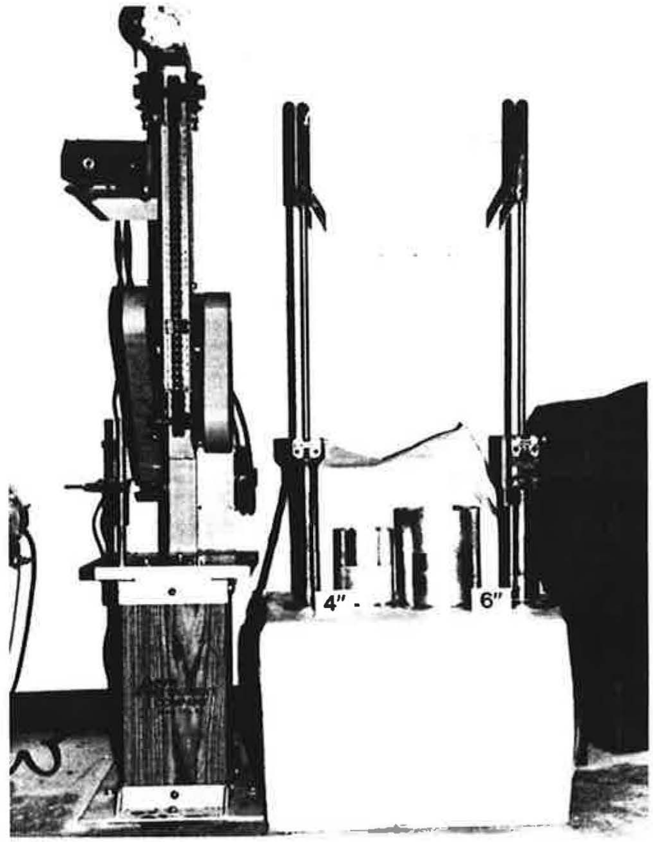

FIGURE 2 Compaction equipment for 4- and 6-in.-diameter specimens.

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

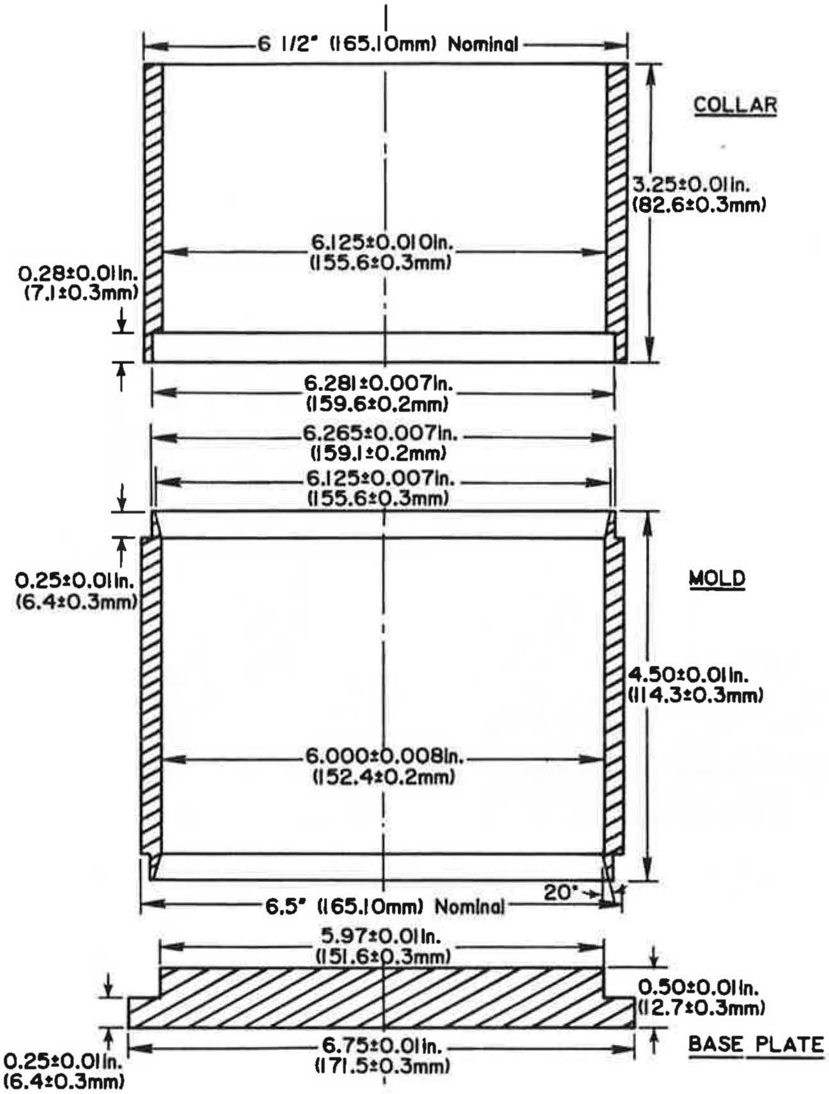

FIGURE 3 Compaction mold.

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

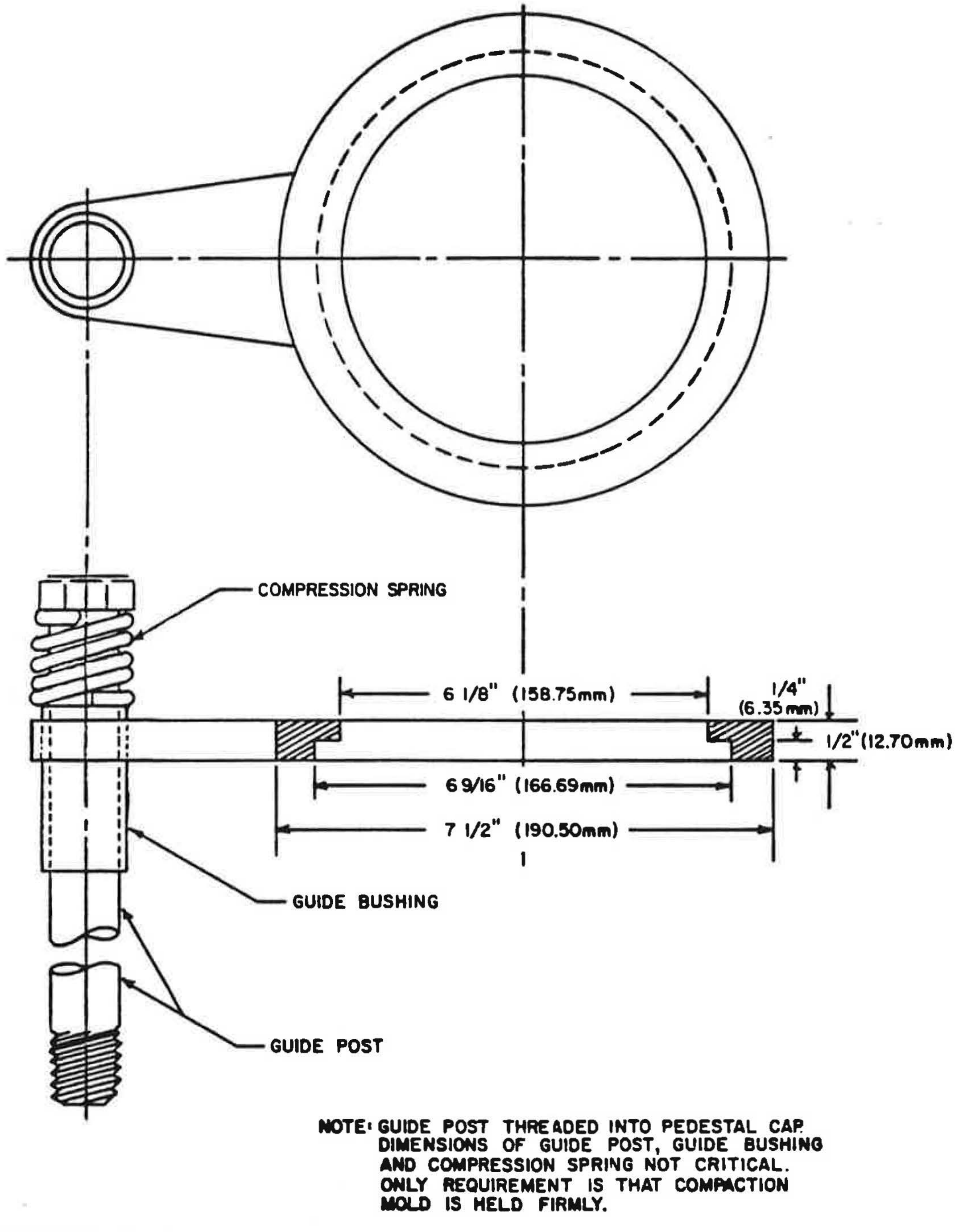

FIGURE 4 Specimen mold holder.

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

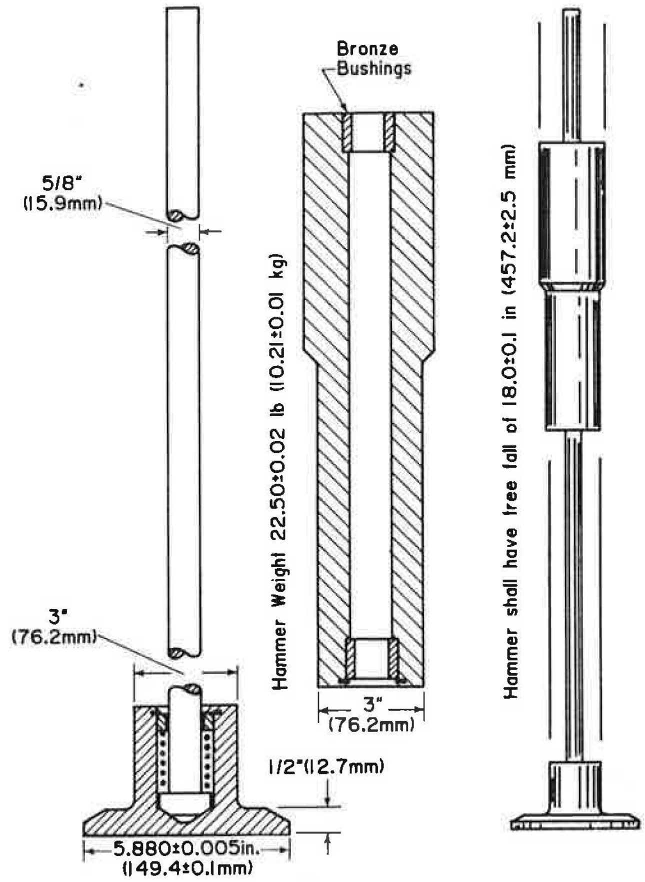

FIGURE 5 Compaction hammer.

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,

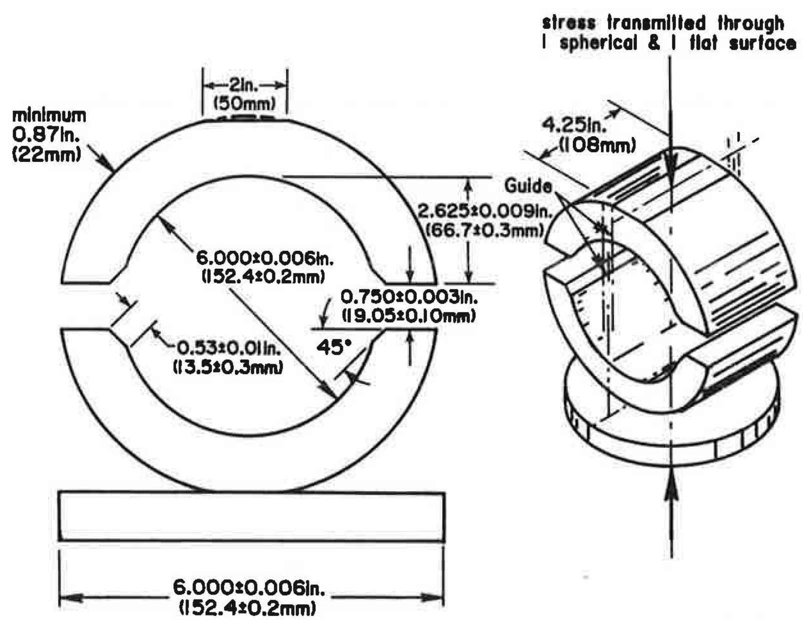

FIGURE 6 Breaking head.

TABLE 7 SUMMARY OF STABILITY AND FLOW RATIOS FOR LARGE-STONE MIXES

Agency (Year data obtained)	No. of Blows 4"	No. of Blows 6"	Ratio Stability	Ratio 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 3.2	3.6	4.0		% Asphalt Content 3.2	3.6	4.0
Bulk Sp. Gr. (1)	2.424	2.410	2.440	Stability (lbs) (1)	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 (units) (1)	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

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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

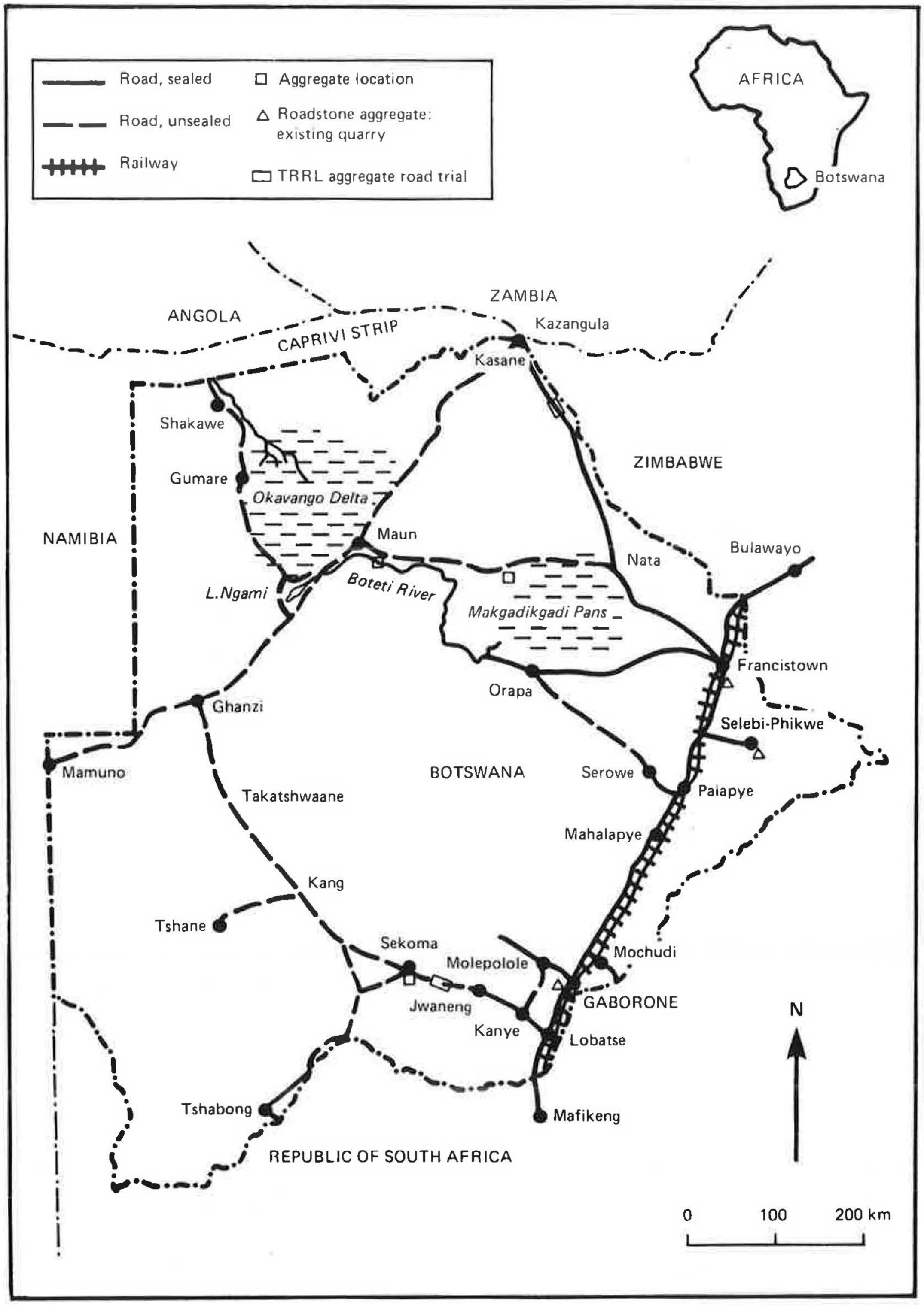


FIGURE 1 Republic of Botswana road network.

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.

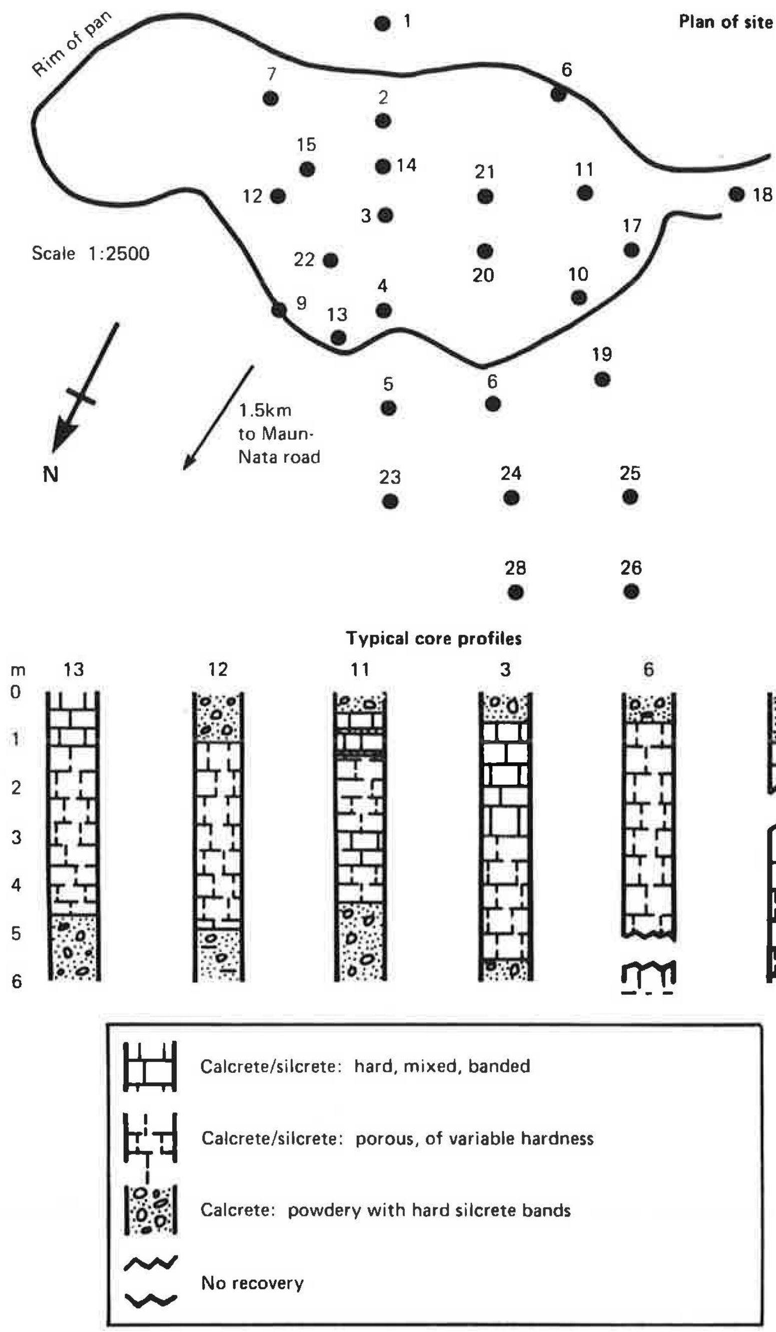


FIGURE 2 Borehole location and typical profiles at MN2 site on Maun-Nata Road.

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	[a]	80	25	–	48	53
Quartz	[a]	12	63	99	35	42
Chalcedony	[a]	7	10	–	–	1
Olivine	[a]	–	–	–	10	–
Mica	[a]	–	–	–	5	–
Other	[a]	1	2	1	2	4

[a]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	[a]
SiO_2 (range)	38–51	4–31	62–94	–	22–77	[a]
Al_2O_3 (median)	1.2	1.2	1.3	1.1	2.0	[a]
Al_2O_3 (range)	0.8–2.4	0–1.4	0.8–5.0	–	3–6	[a]
Fe_2O_3 (median)	0.5	0.5	0.6	0.5	1.3	[a]
Fe_2O_3 (range)	0.5–1.0	0.5–1.7	0.4–8.0	–	1–3	[a]
$CaCO_3$ (median)	39.6	80.1	22.7	0.3	57.6	[a]
$CaCO_3$ (range)	40–87	50–81	1–25	–	5–95	[a]
$MgCO_3$ (median)	10.2	5.6	12.3	0.2	6.0	[a]
$MgCO_3$ (range)	10–50	6–43	3–56	–	1–8	[a]
Na_2O_3 (median)	0.1	0.1	0.6	0.1	0.1	[a]
Na_2O_3 (range)	0–0.1	0.1–0.2	0–1.0	–	–	[a]
K_2O (median)	0.8	0.8	0.6	0.1	0.3	[a]
K_2O (range)	0–0.8	0.2–1.0	0–4.5	–	0.1–0.7	[a]

[a]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.

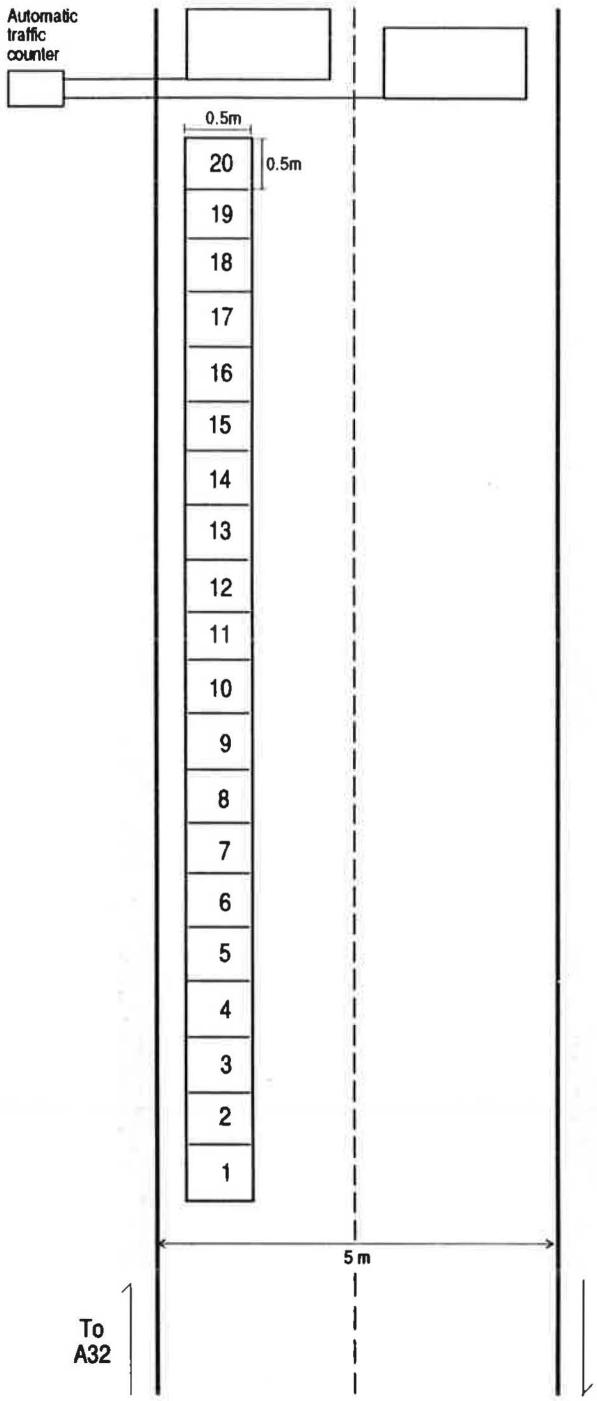
In order to assess the other aggregates included in the study—the MN2 and Samedupe materials—a pilot scale trial was constructed in August 1989 near Winchester in the United Kingdom on a minor road whose premix macadam wearing course was being surface dressed with a single seal. This action was part 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/m²) 1st Seal	Binder Application Rate (L/m²) 2nd Seal	Stone Application Rate (kg/m²) 1st Seal	Stone Application Rate (kg/m²) 2nd Seal	Traffic (1987 veh/day)	Maintenance Treatment	Performance Rating Scale After 5 Years[a]
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	Basalt gravel	Nata calcrete	13 mm, with fog spray	1.6	–	12	–	140 (52% > 5 t.)	Slurry seal (Nov. 1986)	3 to 1
3	Basalt gravel	Nata calcrete	13 mm + crusher waste	1.6	1.2	12	12	140 (52% > 5 t.)	No treatment	2
4	Basalt gravel	Nata calcrete	13 mm + 6.7 mm double	1.6	1.1	12	5	140 (52% > 5 t.)	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	Calcified sand	Sekoma calcrete	19 mm + 6.5 mm	2.1	1.6	19	11	112 (34% > 5 t.)	Fog spray (July 1985)	3
3	Calcified sand	Sekoma calcrete	19 mm + fines	2.7	0.9	19	12	112 (34% > 5 t.)	Fog spray (July 1985)	3–4
7	Calcified sand	Sekoma calcrete	Double graded seal[d]	1.4	1.5	30	24	112 (34% > 5 t.)	Fog spray (July 1985)	3
15	Nodular calcrete	Sekoma calcrete	13 mm + fines	1.6	1.2	12	12	112 (34% > 5 t.)	Fog spray (July 1985)	3
17	Nodular calcrete	Sekoma calcrete	19 mm + 9.5 mm	1.8	1.4	19	11	112 (34% > 5 t.)	Fog spray (July 1985)	4
18	Nodular calcrete	Dolerite[c]	19 mm + 9.5 mm	1.6	2.0	19	11	112 (34% > 5 t.)	Fog spray (July 1985)	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.



PLAN OF ROAD

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 /m^2
Stone application rate: 10kg/m^2
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

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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

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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 surficial 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

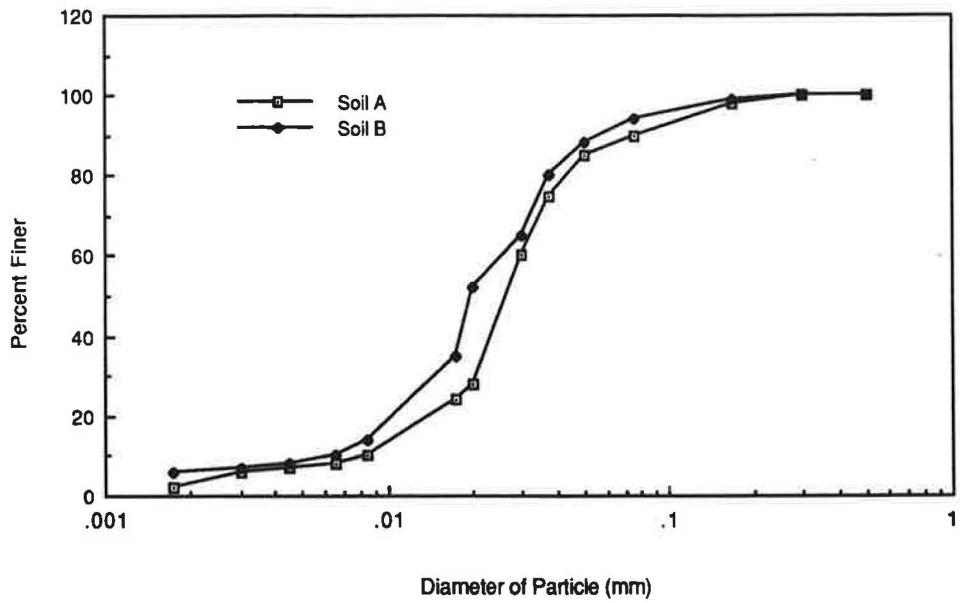

FIGURE 1 Grain size distribution curves of tested soils.

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

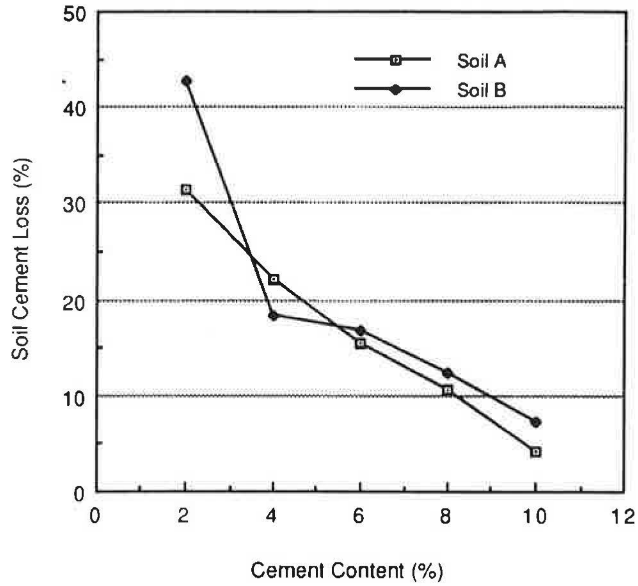

FIGURE 2 Effect of admixture content on soil-cement loss of soils in wet-dry test.

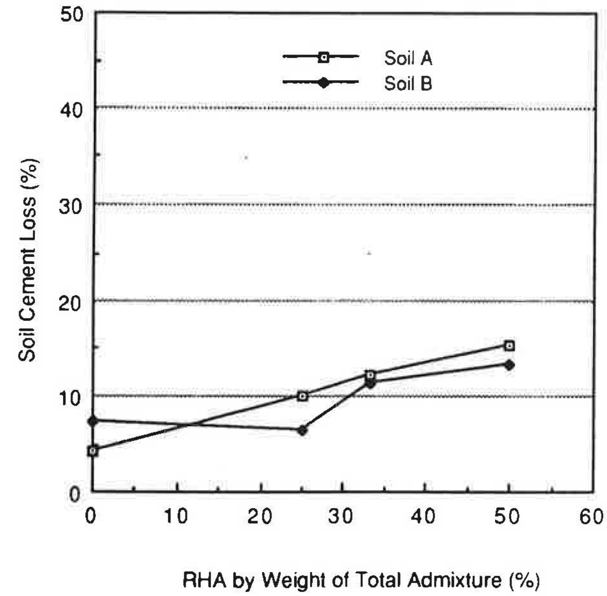

FIGURE 3 Effect of replacement of cement by RHA in the total admixture on soil-cement loss of soils.

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

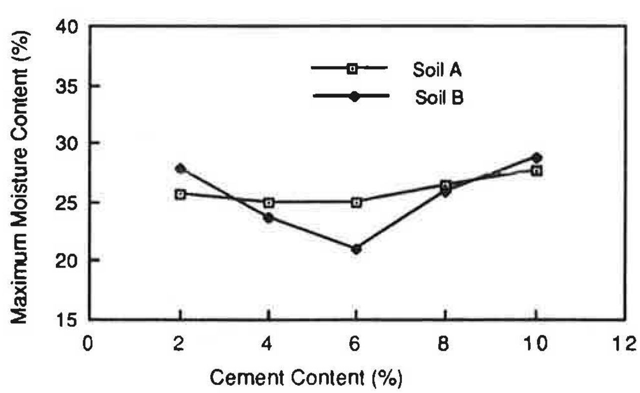

FIGURE 4 Maximum moisture content in cement-stabilized soils during wet cycles of wet-dry test.

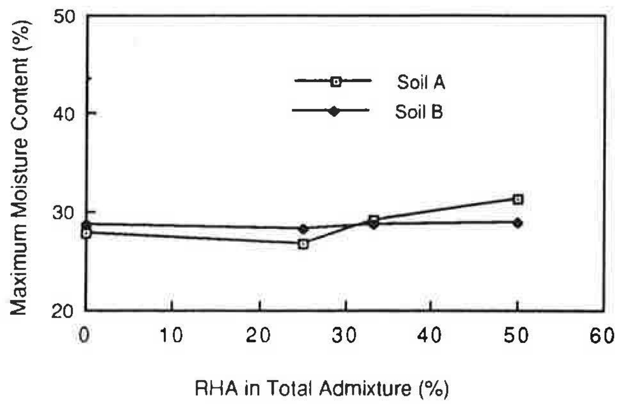

FIGURE 5 Maximum moisture content in cement–RHA-stabilized soils during wet cycles of wet-dry test.

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

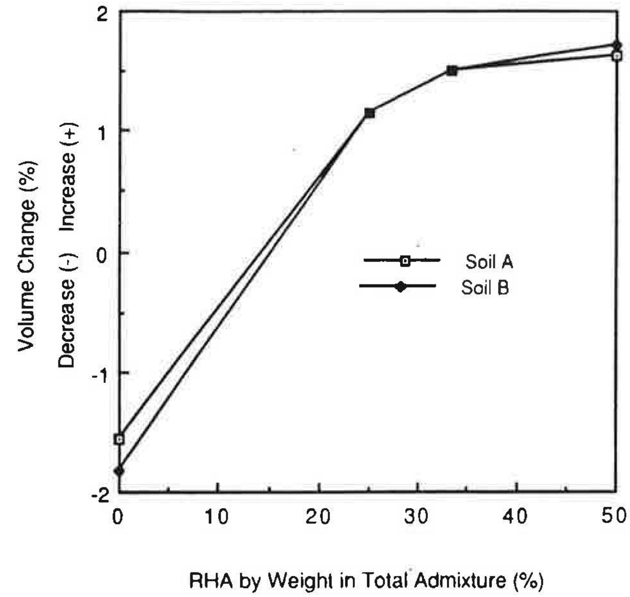

FIGURE 7 Volume change of cement-treated soils on addition of RHA in wet-dry test.

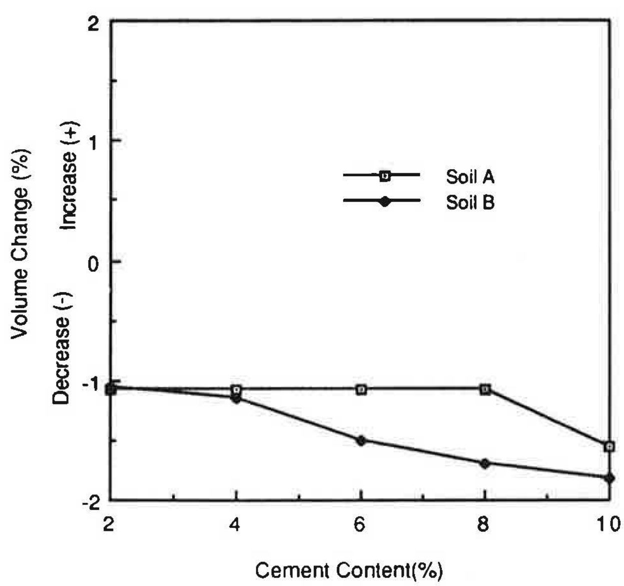

FIGURE 6 Volume change of cement-treated soils in wet-dry test.

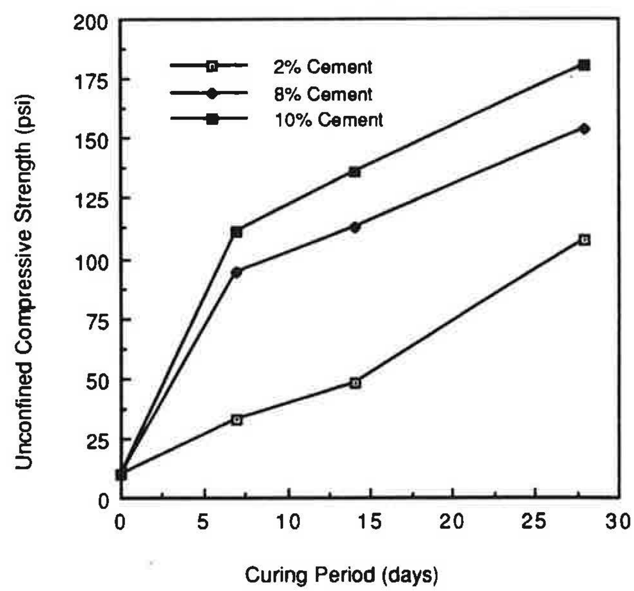

FIGURE 8 Effect of admixture content and curing period on compressive strength of Soil A.

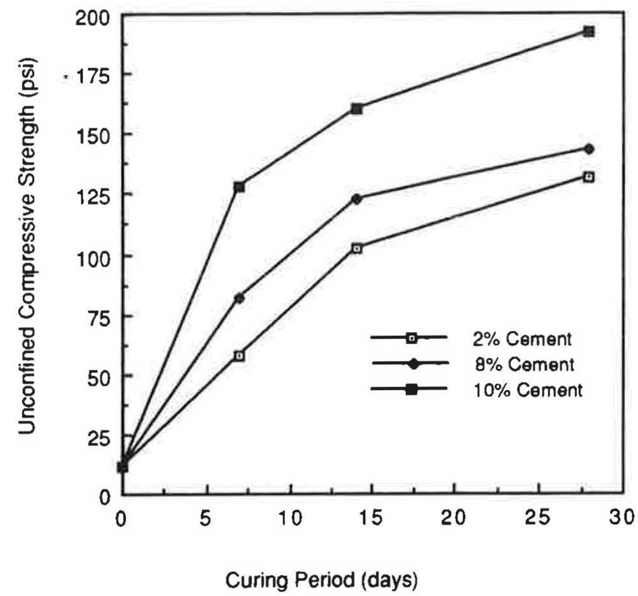

FIGURE 9 Effect of admixture content and curing period on compressive strength of Soil B.

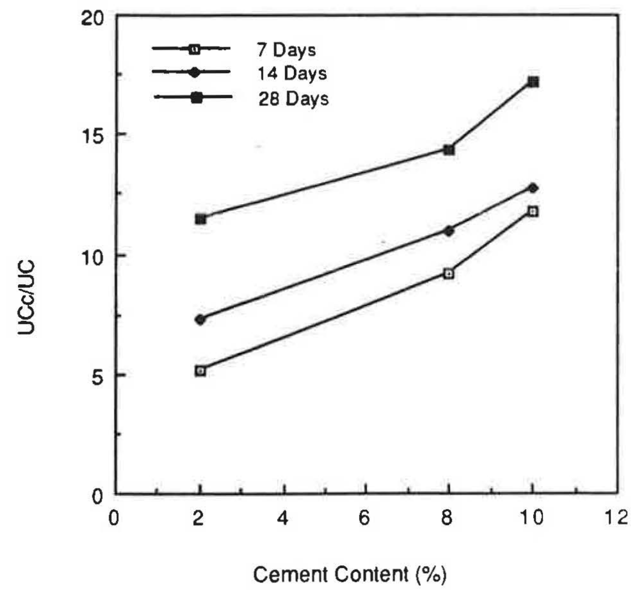

FIGURE 10 Strength ratio of cement-treated Soil A to untreated soil.

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

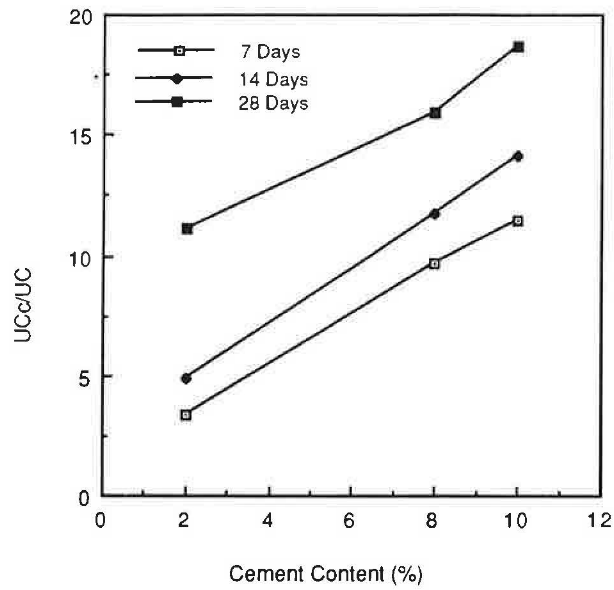

FIGURE 11 Strength ratio of cement-treated Soil B to untreated soil.

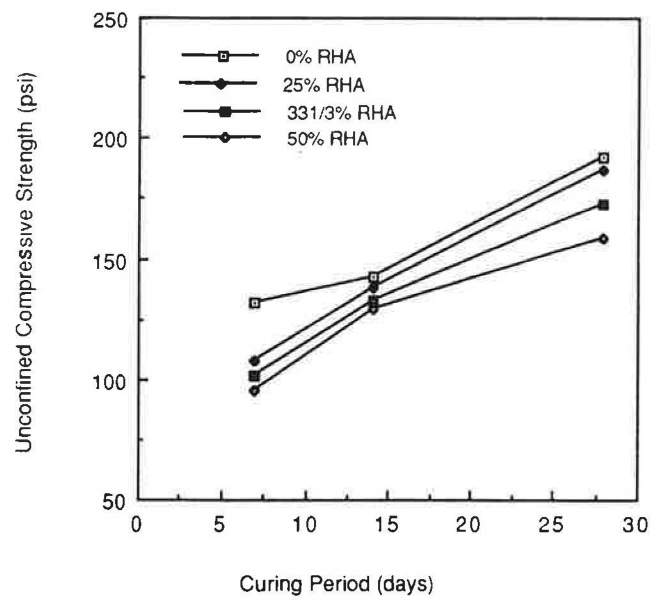

FIGURE 13 Effect of RHA replacement in cement on compressive strength of cement–RHA-treated Soil B.

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).

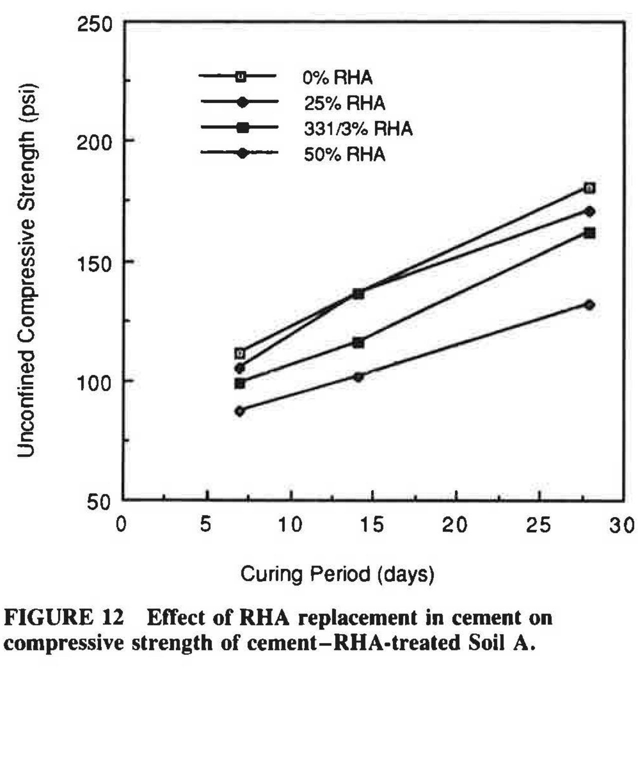

FIGURE 12 Effect of RHA replacement in cement on compressive strength of cement–RHA-treated Soil A.

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 Sample	Cement: RHA	UC_{RHA-c}/UCc 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-

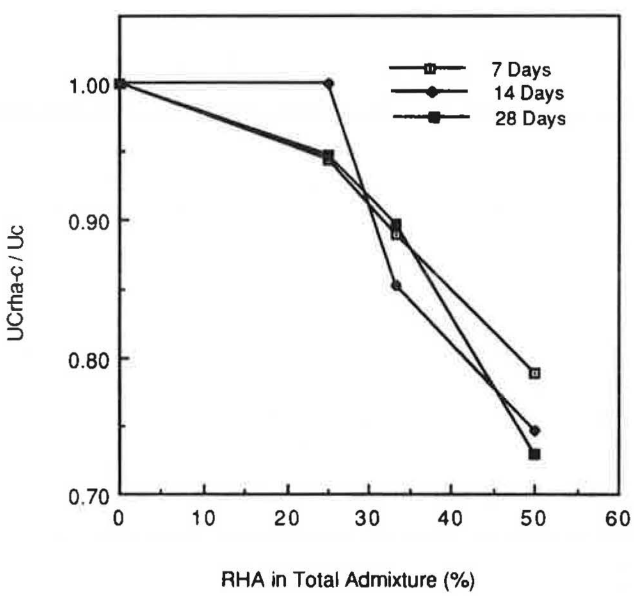

FIGURE 14 Strength ratio of RHA–cement-treated soil to cement-treated soil versus RHA content in total admixture for Soil A.

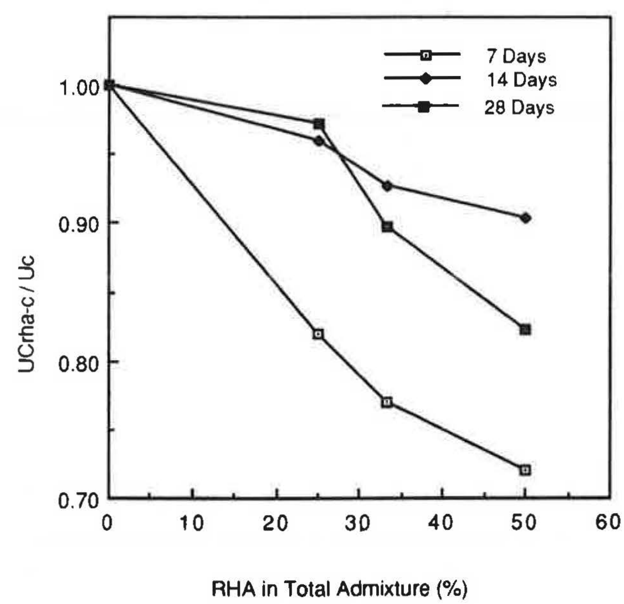

FIGURE 15 Strength ratio of RHA–cement-treated soil to cement-treated soil versus RHA content in total admixture for Soil B.

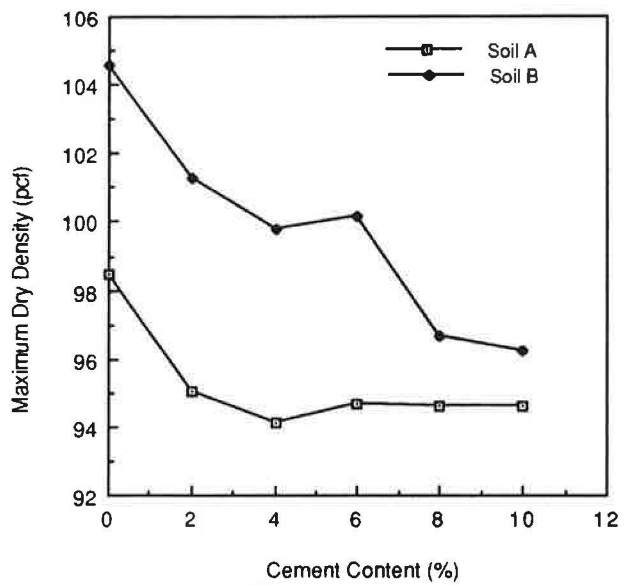

FIGURE 16 Effect of cement addition on dry densities of Soils A and B.

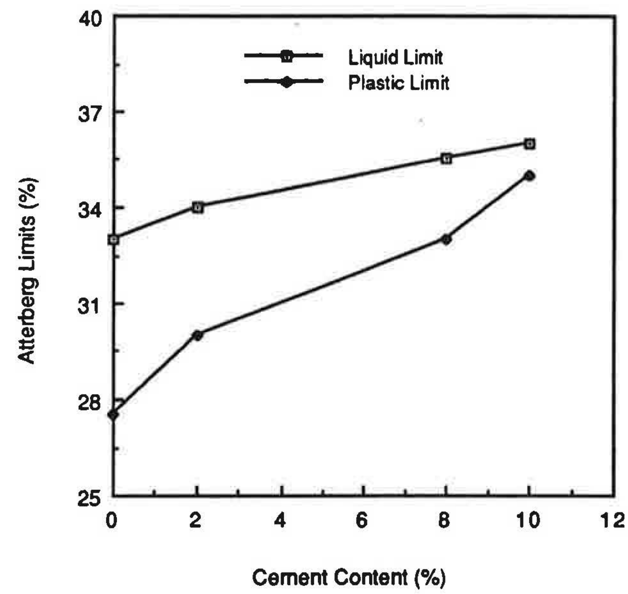

FIGURE 18 Effect of cement addition on Atterberg limits of Soil B.

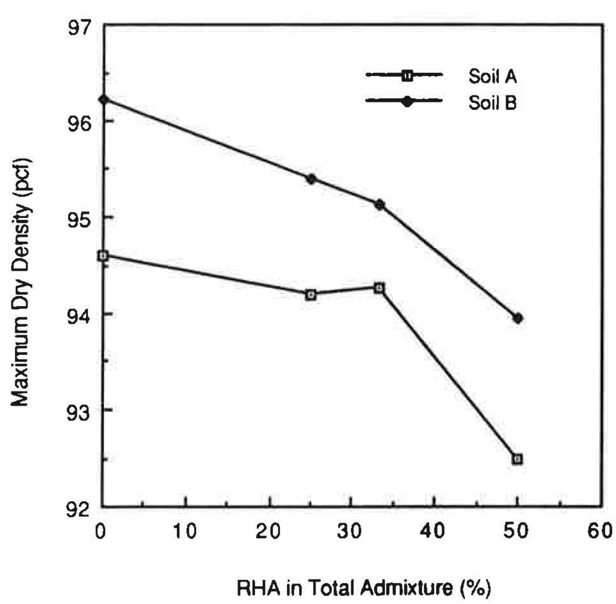

FIGURE 17 Effect of RHA addition on dry densities of cement-treated Soils A and B.

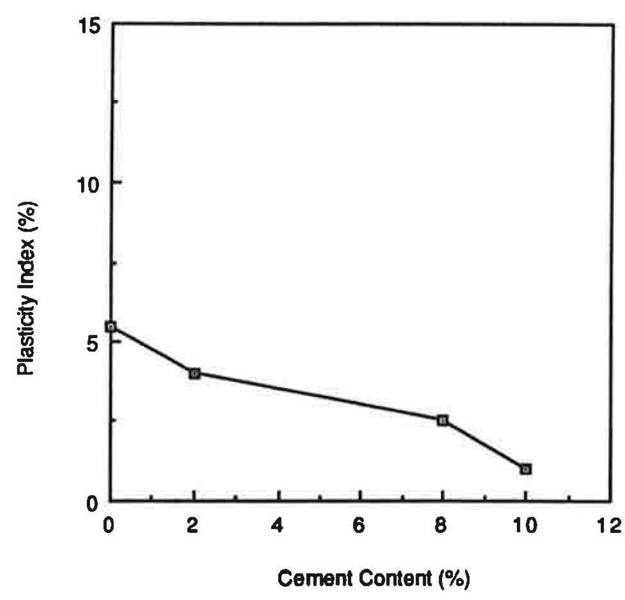

FIGURE 19 Effect of cement addition on plasticity index of Soil B.

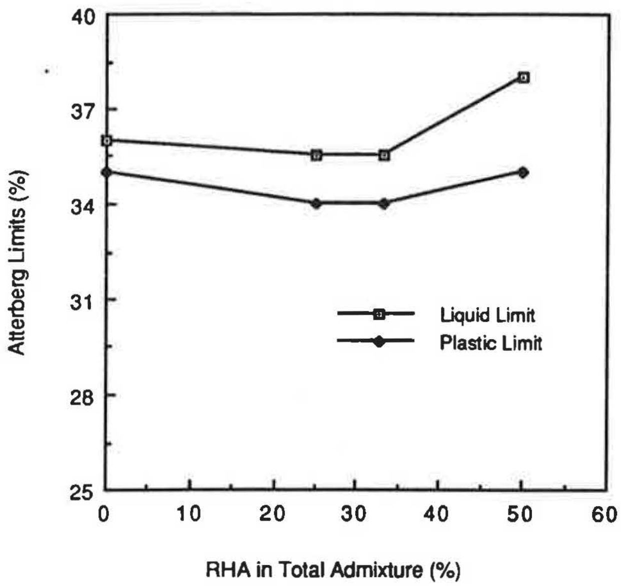

FIGURE 20 Effect of RHA-cement addition on Atterberg limits of Soil B.

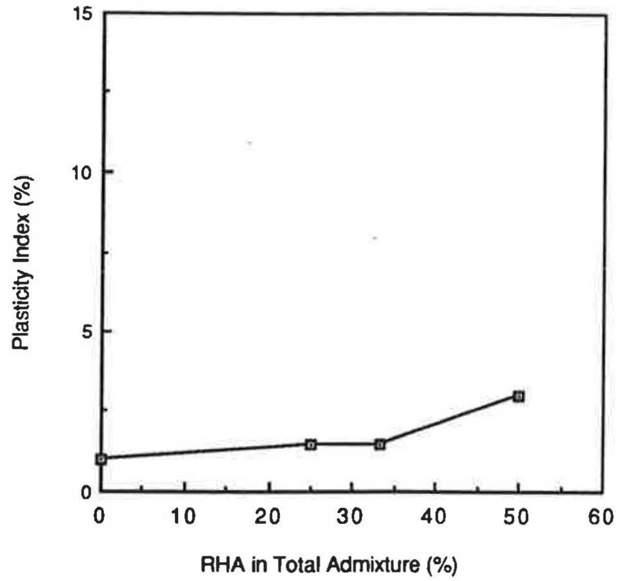

FIGURE 21 Effect of RHA-cement addition on plasticity index of Soil B.

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

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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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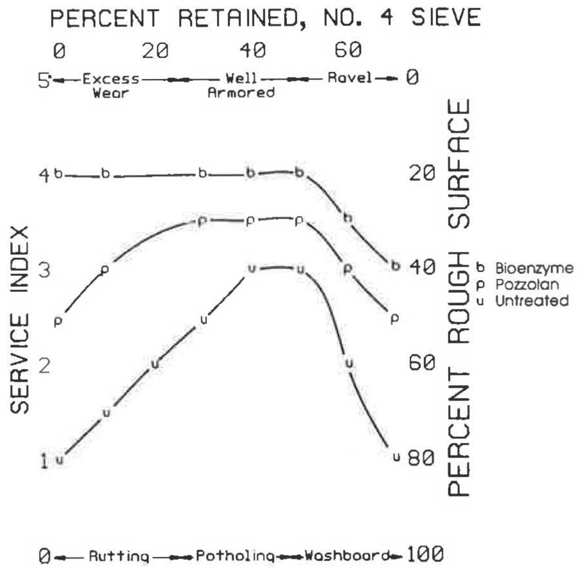

FIGURE 1 Performance of treated surfaces as related to percentage of coarse material in aggregate.

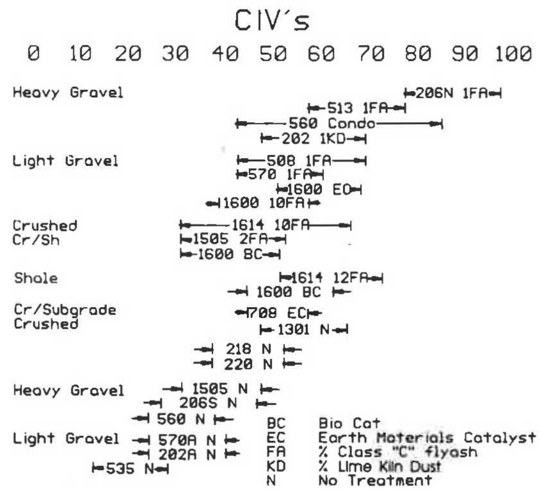

FIGURE 2 CIVs for untreated and treated aggregate surfaces.

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.

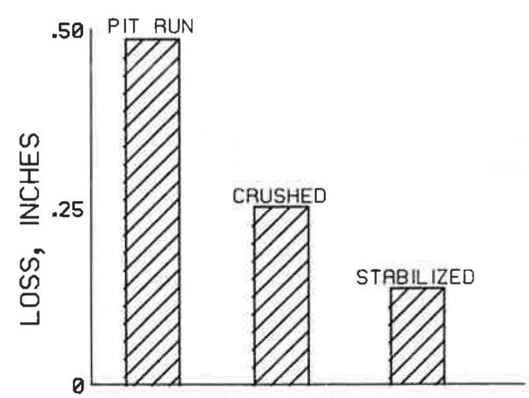

FIGURE 3 Comparison of aggregate loss for treated aggregate, untreated crushed aggregate, and pit run aggregate surfaces.

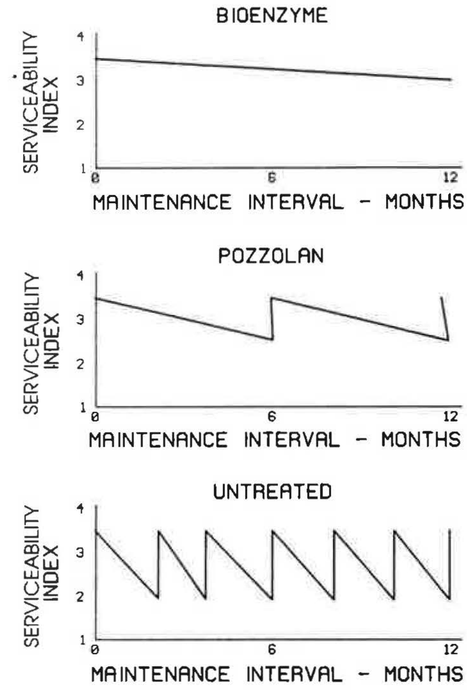

FIGURE 4 Serviceability index comparison for treated and untreated aggregate surfaces.

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

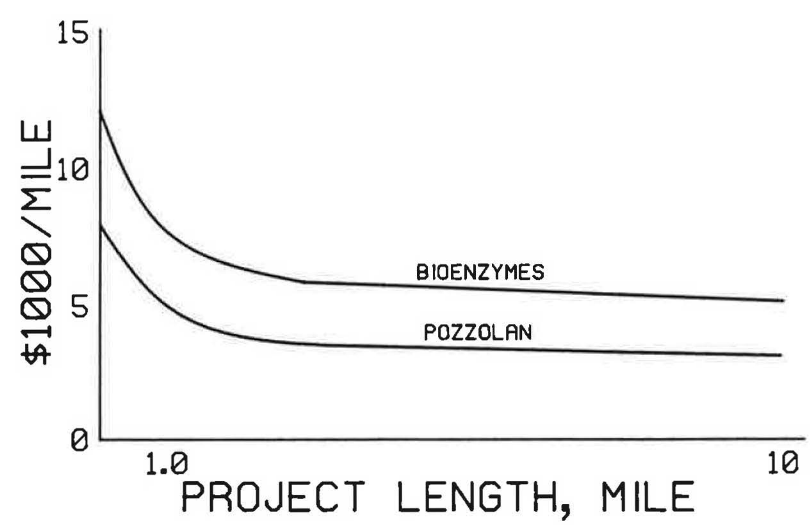

FIGURE 5 1988 construction costs for aggregate surface treatment with nonstandard stabilizers.

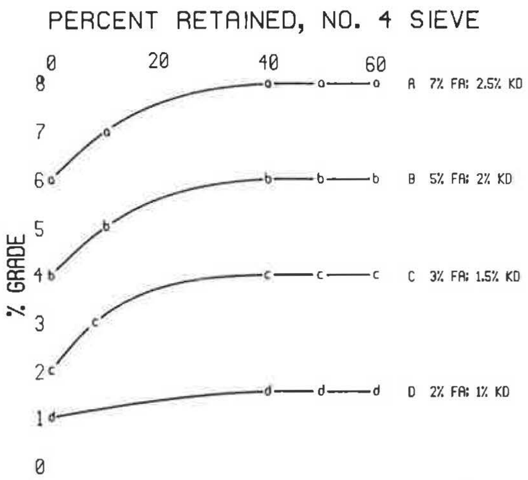

FIGURE 6 Estimated minimum percentages of Class C fly ash and kiln dust required to prevent corrugations on grades.

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

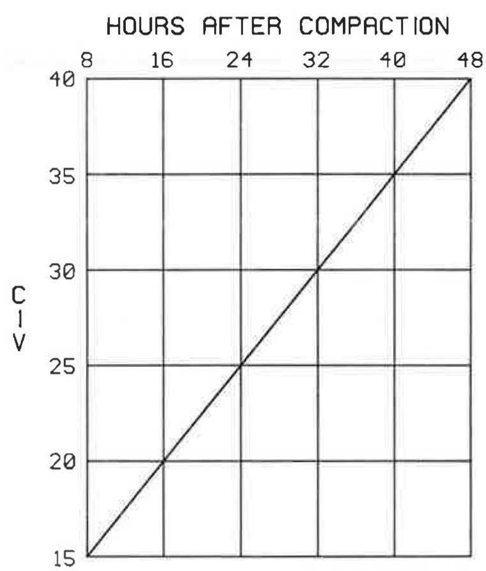

FIGURE 7 Rate of curing for bioenzyme stabilizers, expressed as a function of CIVs.

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.

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. Permazyme 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);

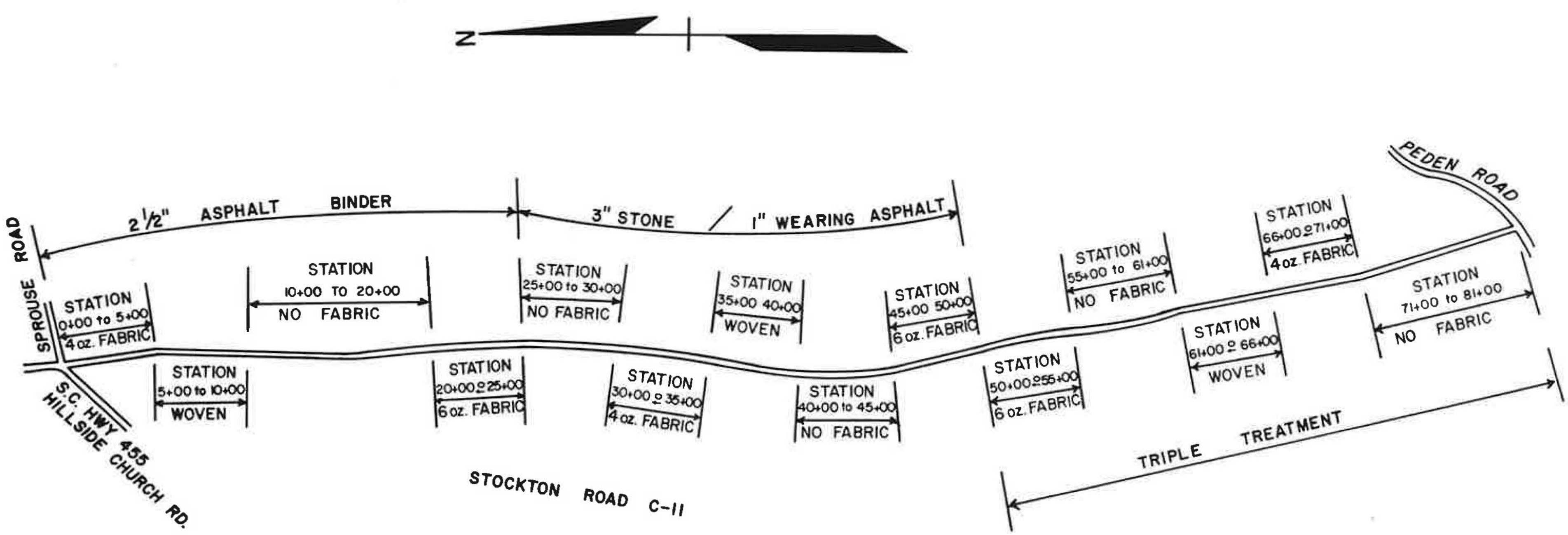

FIGURE 1 Stockton Road experimental paving plan of May 11, 1987.

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

		A	B	C
Construction	ASTM Method	PET Continuous Filament Needlepunched Nonwoven	PP Silt Film Woven	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 Sub-grade)	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 con- nected Alligator Patterns	Slight Trans- inverse and Long- tudinal Cracking	More Hairline Cracking	Slight Trans- verse and Long- tudinal Cracking	Same	Slight Longitu- dinal Cracking	More Trans- verse Cracking	Same	More Alligator Cracking	Severe Alligator Cracking	Same	Same	Slight Alligator and Longitu- dinal Cracking
Feb. 15, 1989	Same	Extensive Alligator Cracking Repress- ing Into Large Block Cracking	Repaired Ravelling Intermit- tent Trans- verse and minor Long. Cracking	Hairlines are now Signifi- cant Pattern of Trans- verse and Long Cracks	More Trans- verse and Long tudinal Cracking	Few Minor Longitu- dinal Cracks	Slight Longitu- dinal Cracking	Intermit- tent Block Cracking	Same	More Alligator Cracking	Alligator Cracking Removed and patched	Same	Same	More Alligator and Longitu- dinal Cracking
July 21, 1989	Minor Trans- verse Crack	Most Severe Alligator Patched, Blocks becoming Alligator	Intermit- tent Cracking Beginning to Block	Alligator Cracking Developin from tran verse & Longi- tudinal	Large Block Cracking Develop- ing	Minor Cracks Appear to have healed in hot Weather	Most Cracks Appear to have healed in hot Weather	Increas- ing Inter- mittent Block Cracking	Minor Longitu- dinal Crack	Alligator Cracking Stabili- zed	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/yd^2 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/yd^2 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/yd^2 woven and nonwoven fabrics performed equally as well in this installation, whereas the 6-oz/yd^2 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/yd^2 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/yd^2 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 Cracking, 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.

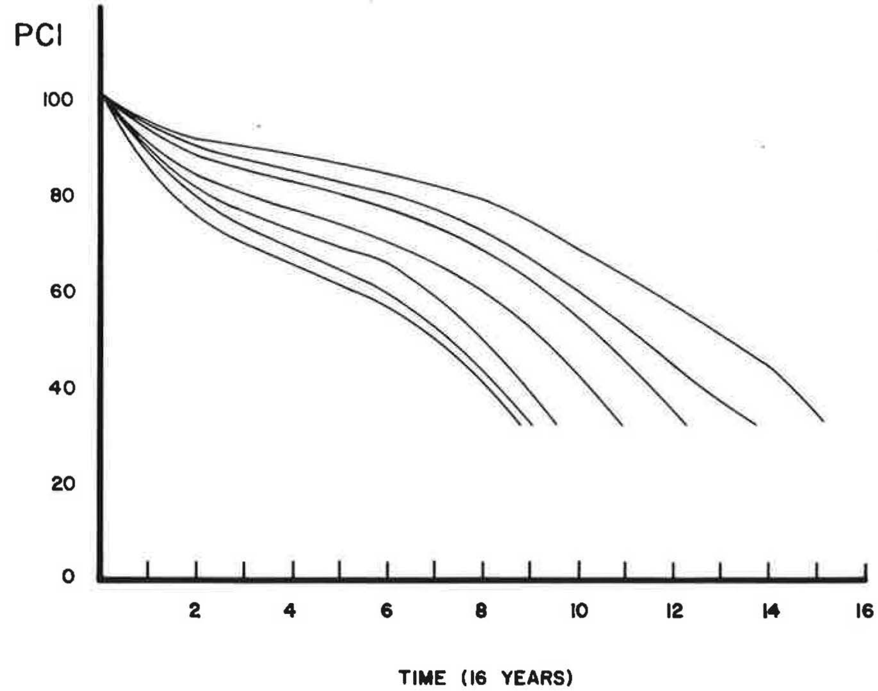

FIGURE 2 Predicted PCI family of curves.

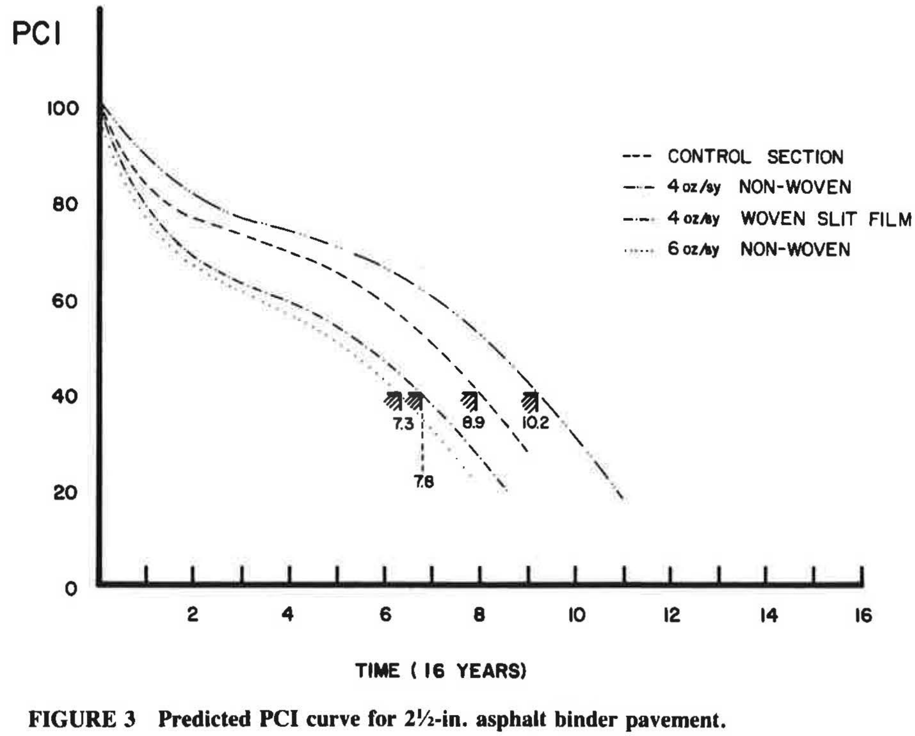

FIGURE 3 Predicted PCI curve for 2½-in. asphalt binder pavement.

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.

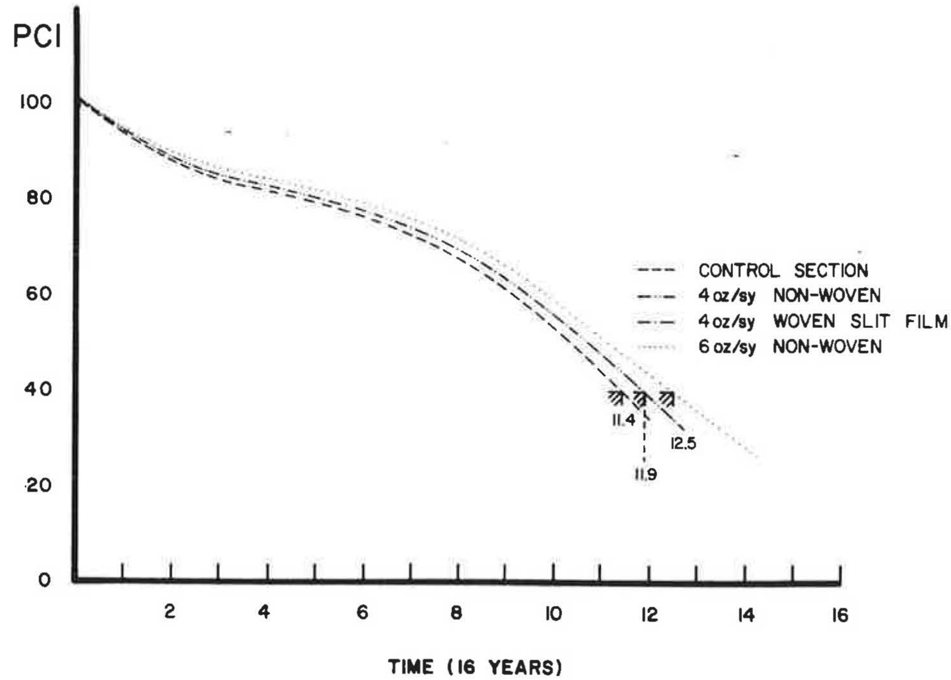

FIGURE 4 Predicted PCI curve for 1½-in. asphalt wearing course over 3-in. stone base.

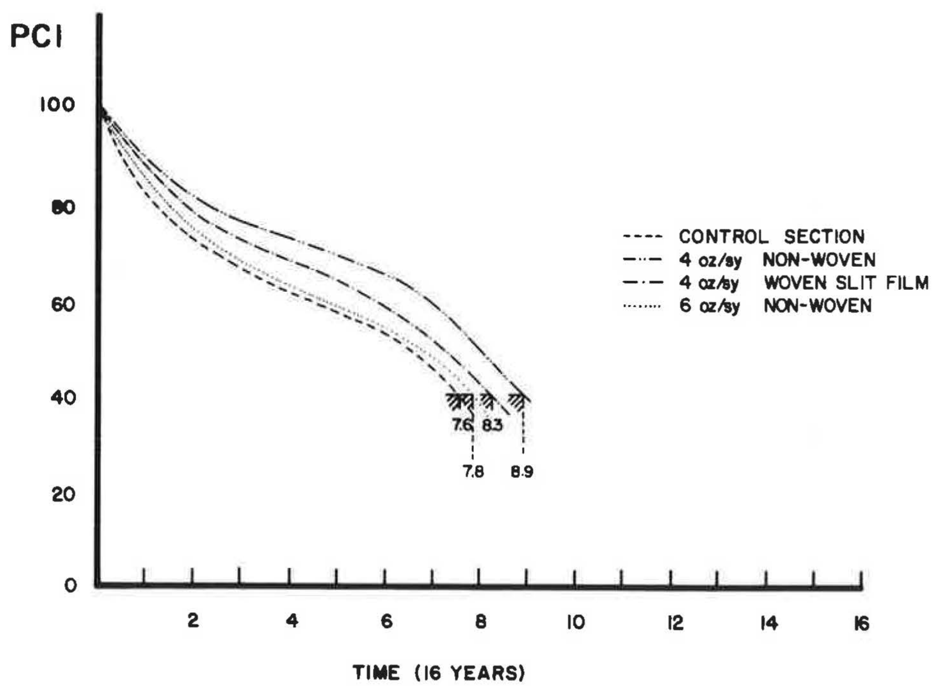

FIGURE 5 Predicted PCI curve for triple treatment over 3-in. stone base.

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.

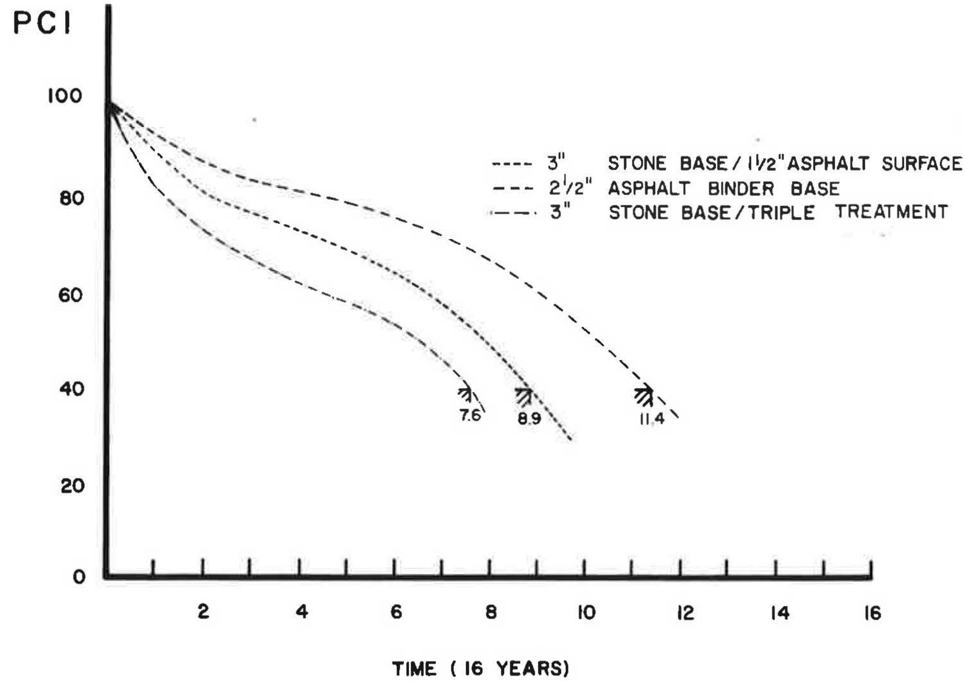

FIGURE 6 **Predicted PCI curve for control pavement sections.**

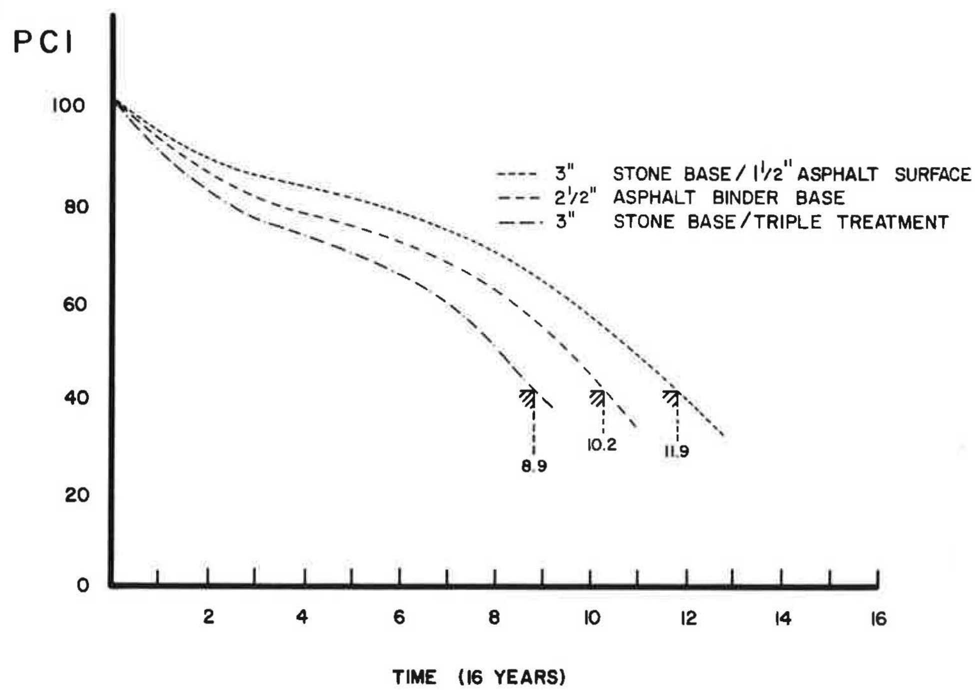

FIGURE 7 **Predicted PCI curve for pavements using 4-oz/yd² nonwoven fabrics.**

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.

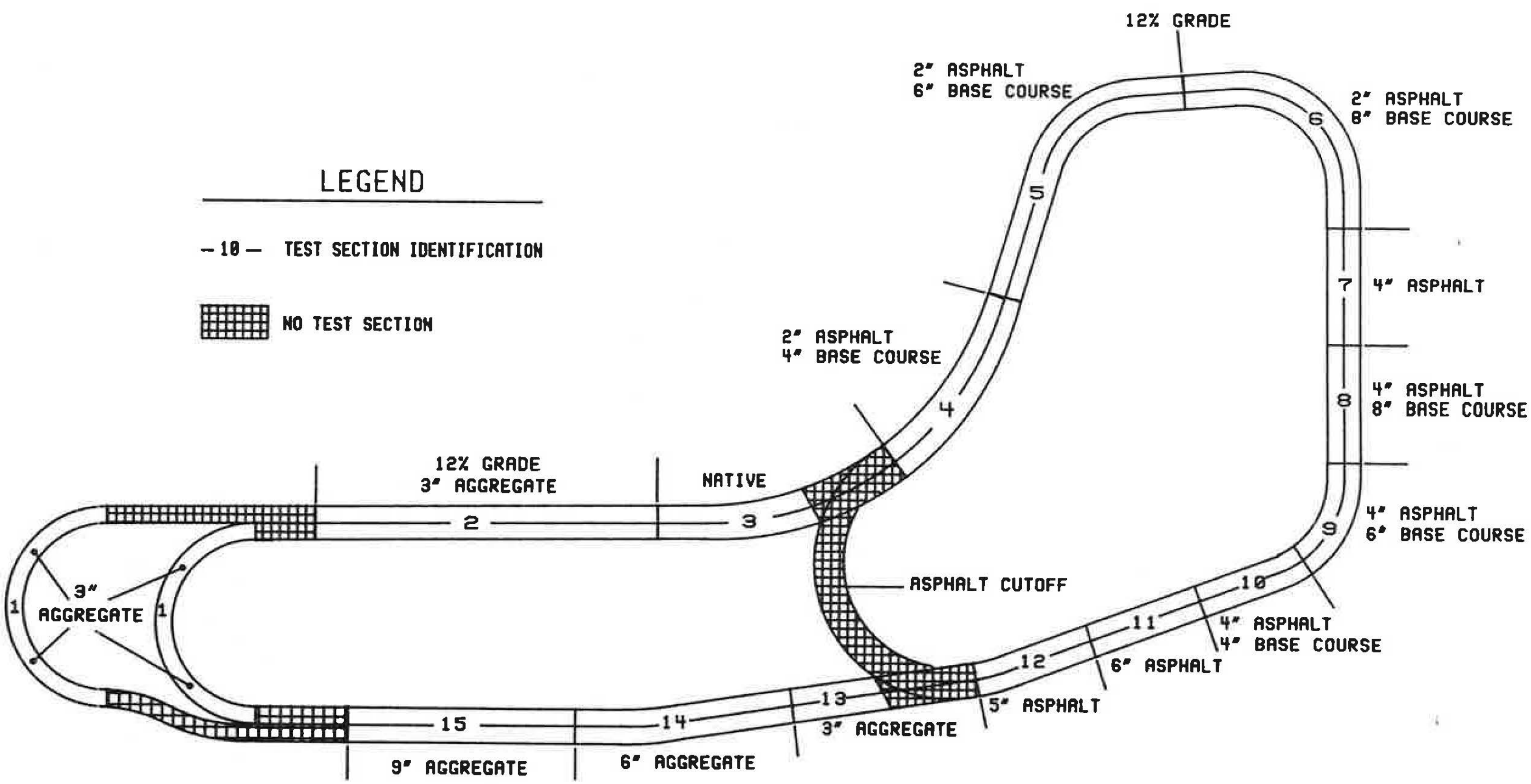
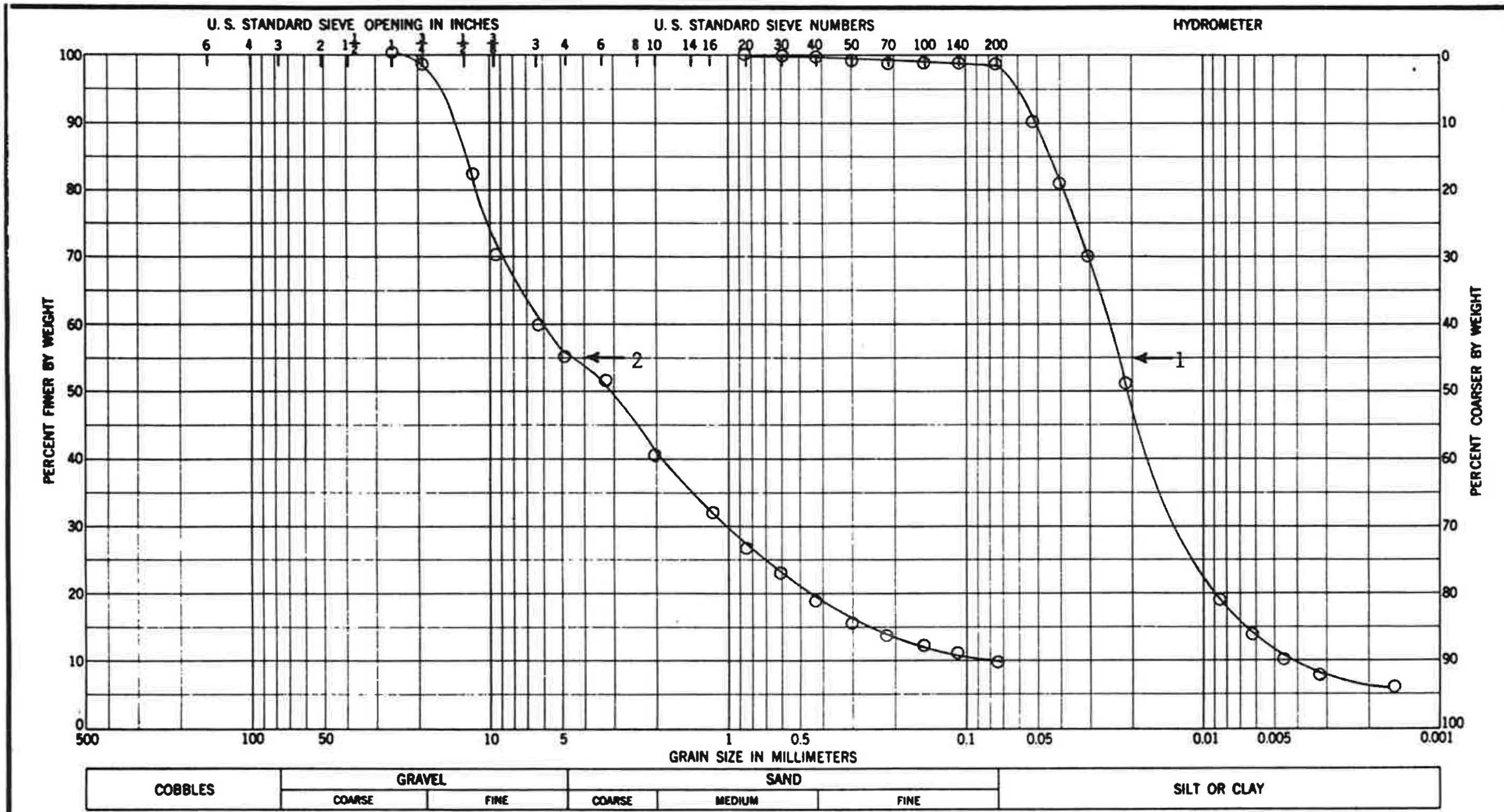


FIGURE 1 **Plan view of test road.**



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

Project

Area

Boring No.

Date

GRADATION CURVES

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FIGURE 2 Classification data for subgrade and base materials.

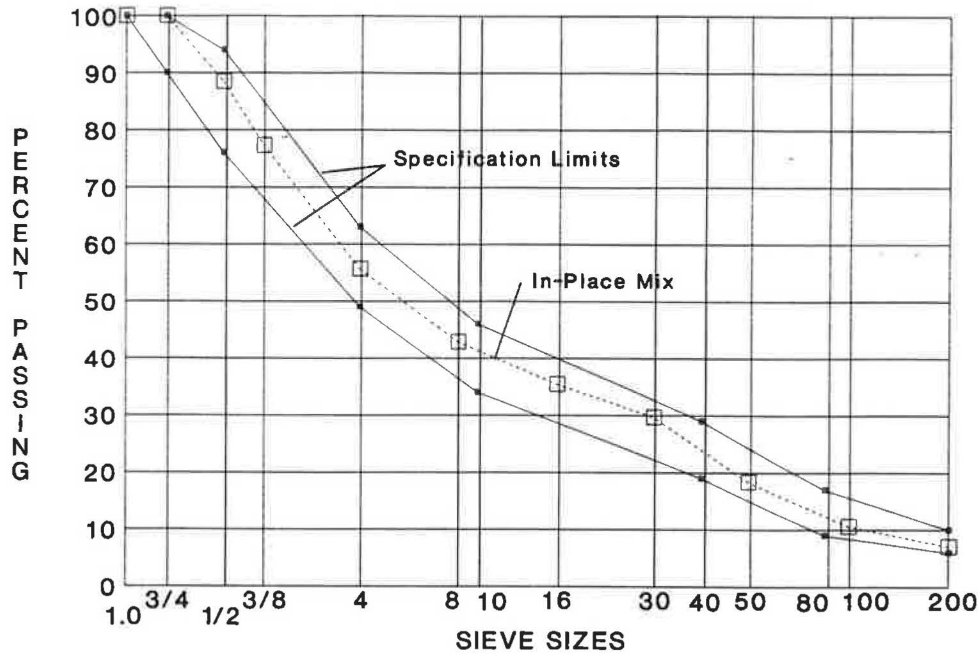

FIGURE 3 Combined gradation curve and gradation specification limits for AC.

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	Actual	Passes	Depth (in.)	CBR (%)	Water Content (%)	Dry Density, pcf In-Place A	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 Loaded	Unloaded	Low Pressure Lane, passes 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

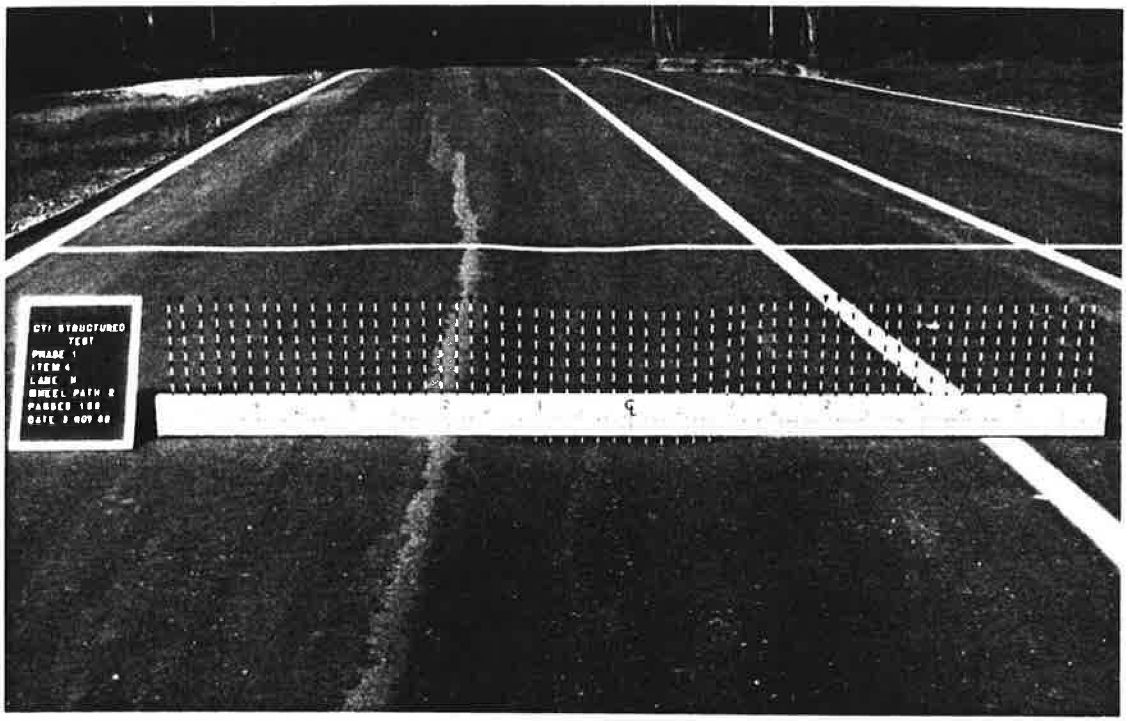

FIGURE 5 High-pressure lane, Section 4, general view of initial failure, 158 passes.

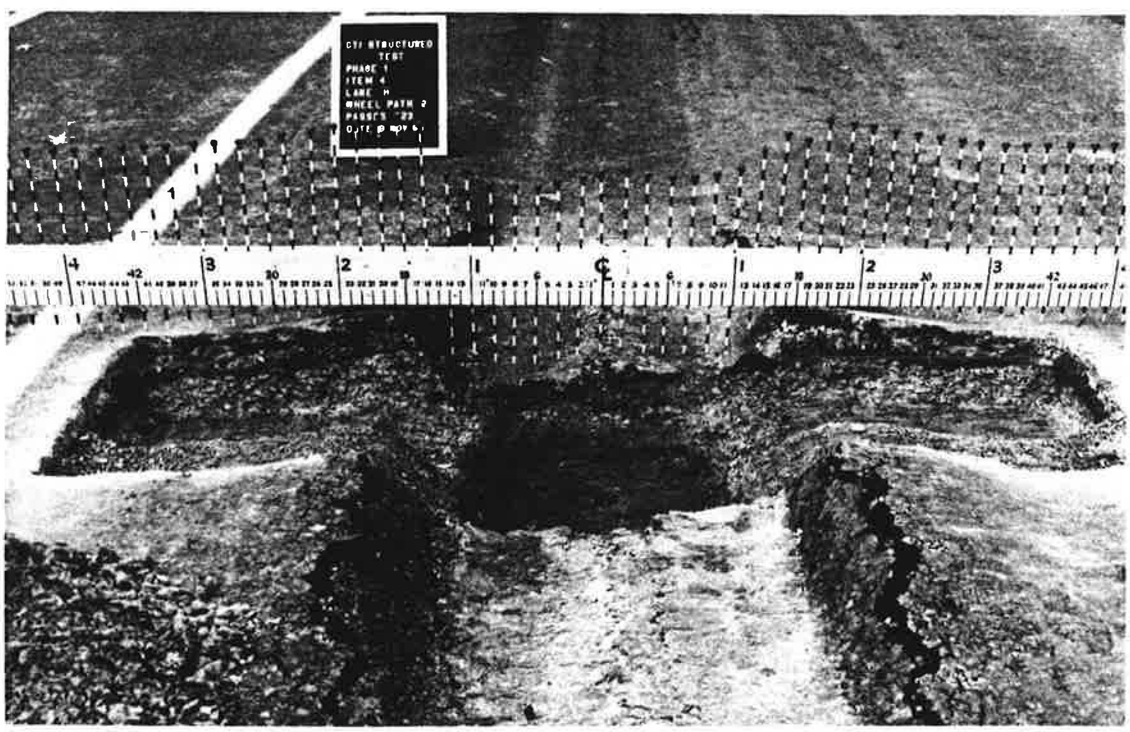

FIGURE 6 High-pressure lane, Section 4, test trench after 323 passes.

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

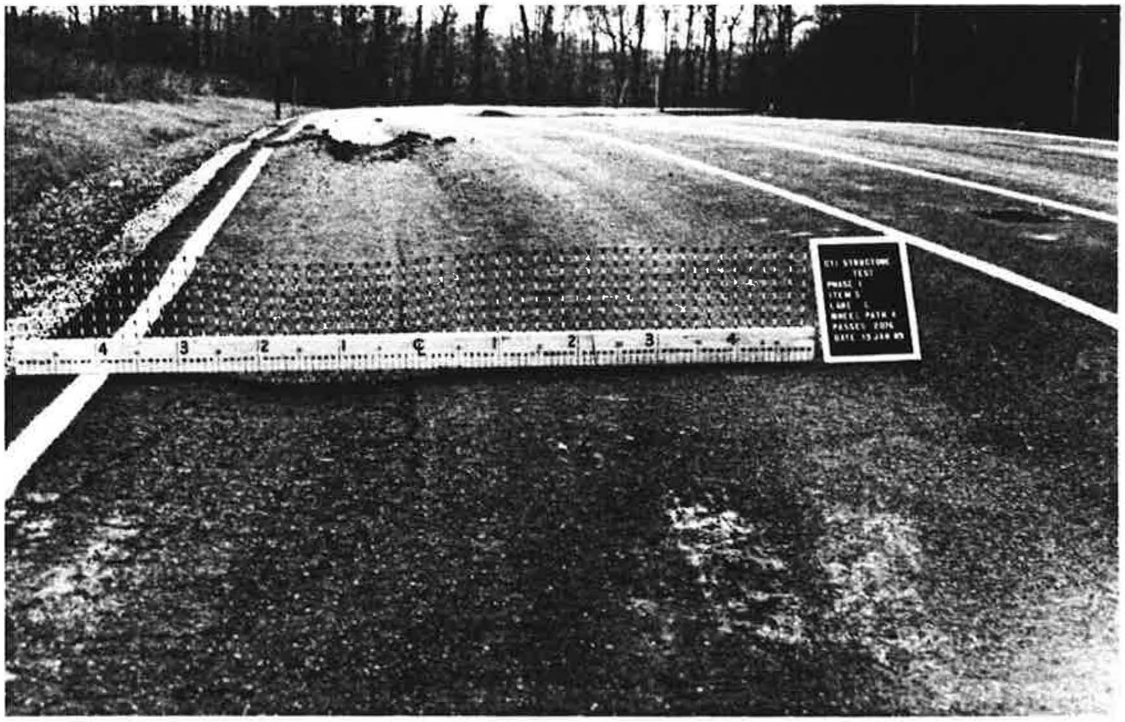

FIGURE 9 Low-pressure lane, Section 5, failure after 2,076 passes.

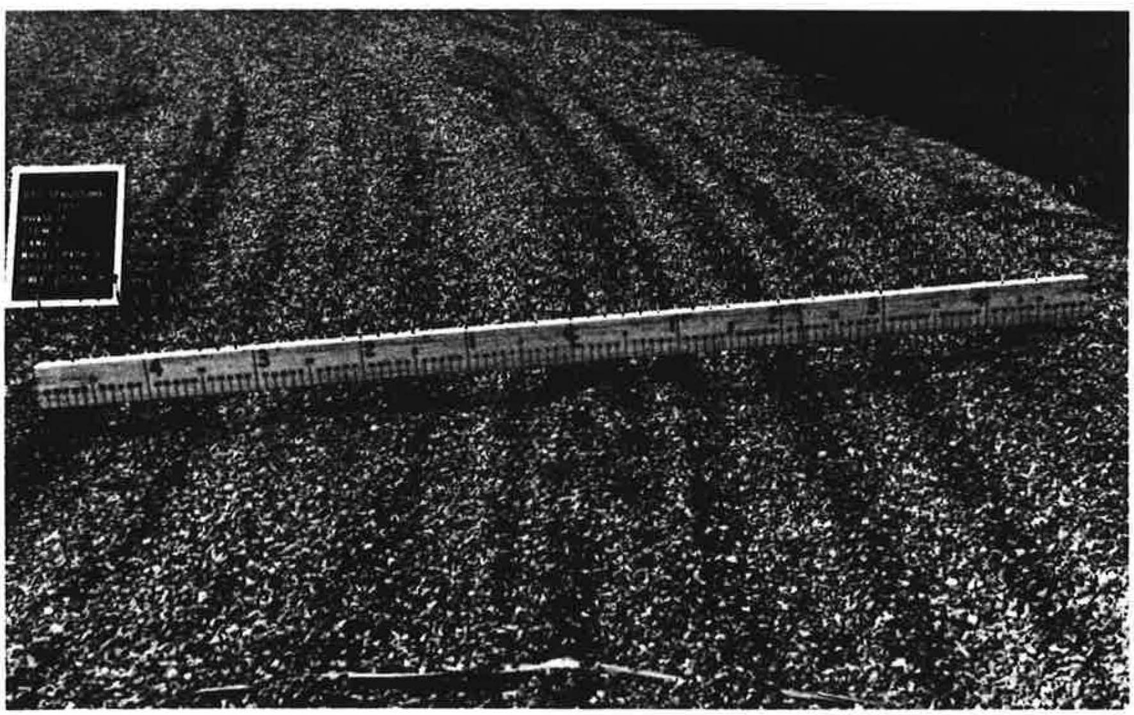

FIGURE 10 Low-pressure lane, Section 1, failure after 66 passes.

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.

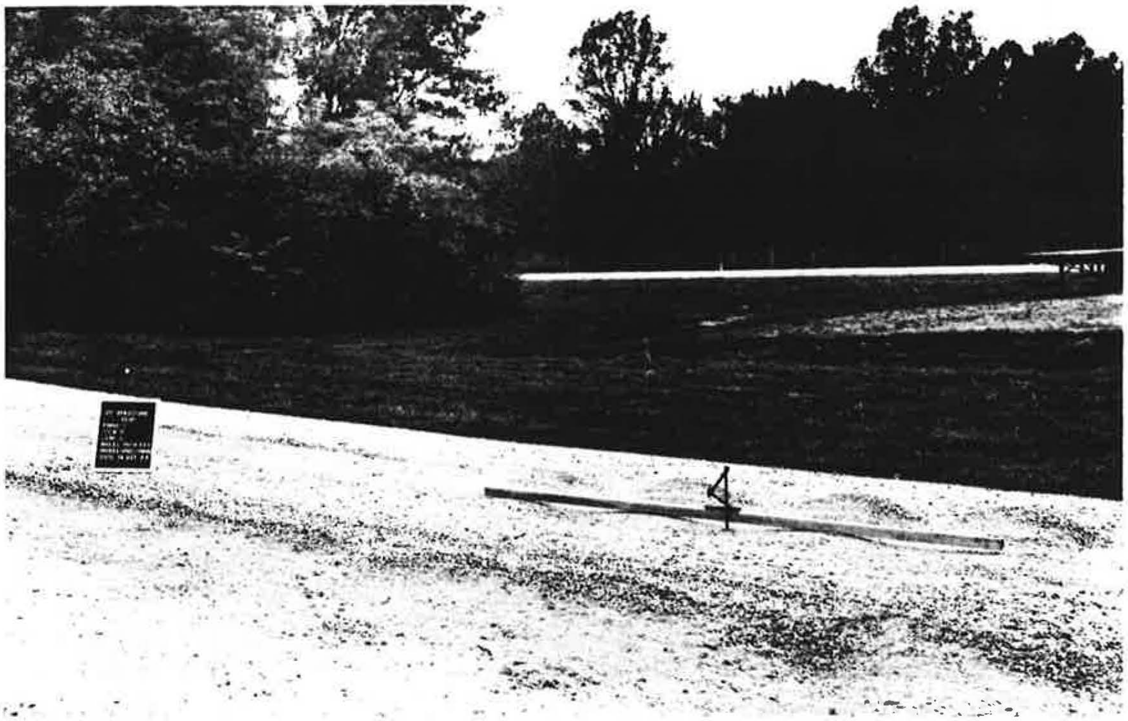

FIGURE 12 Low-pressure lane, Section 2, long intervals between corrugations.

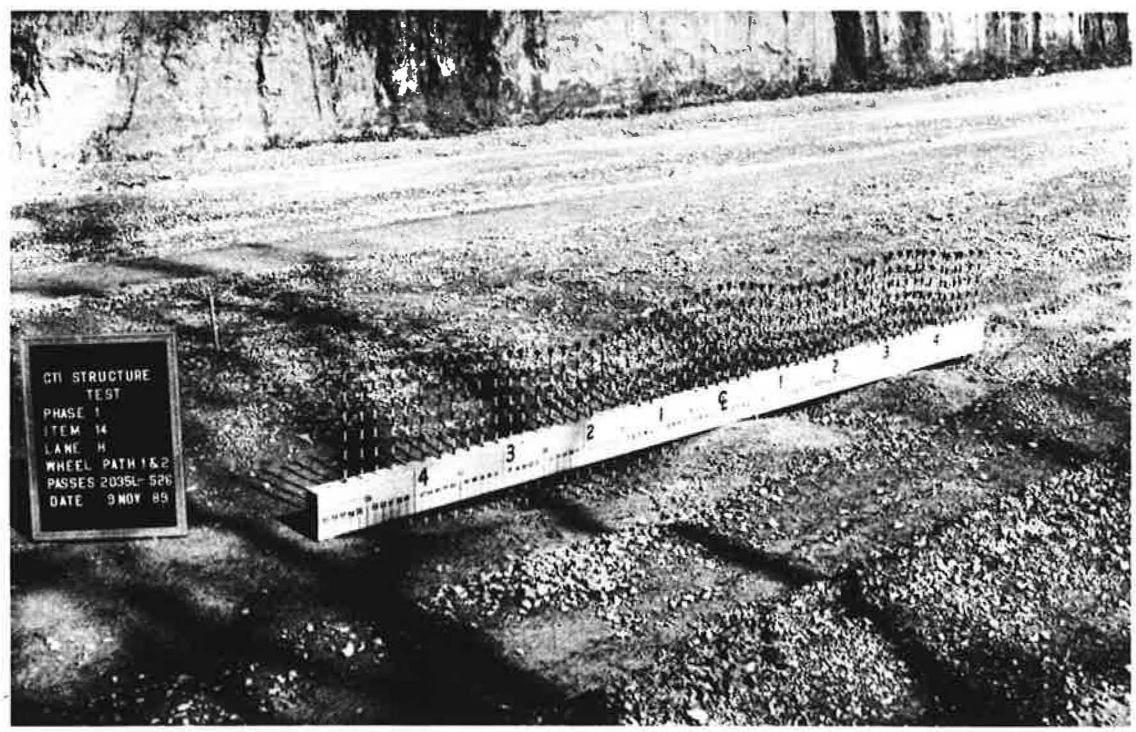

FIGURE 13 High-pressure lane, Section 14, high-severity washboarding.

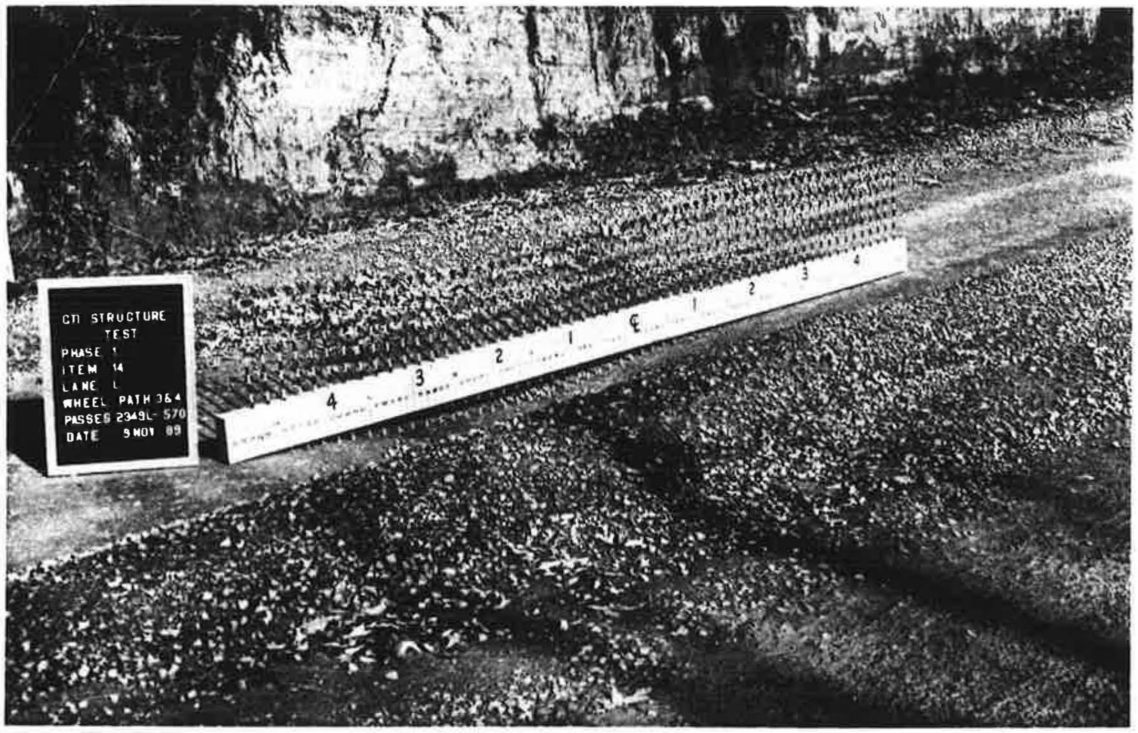

FIGURE 14 Low-pressure lane, Section 14, low-severity depression.

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

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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 yd³ 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

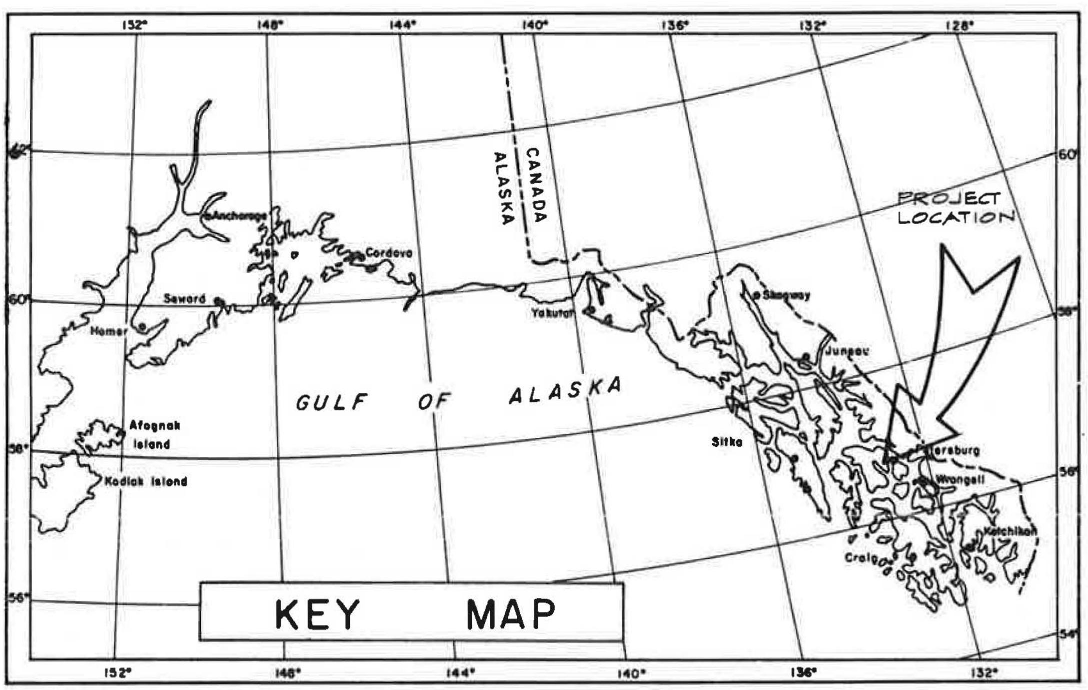

FIGURE 4 Key map.

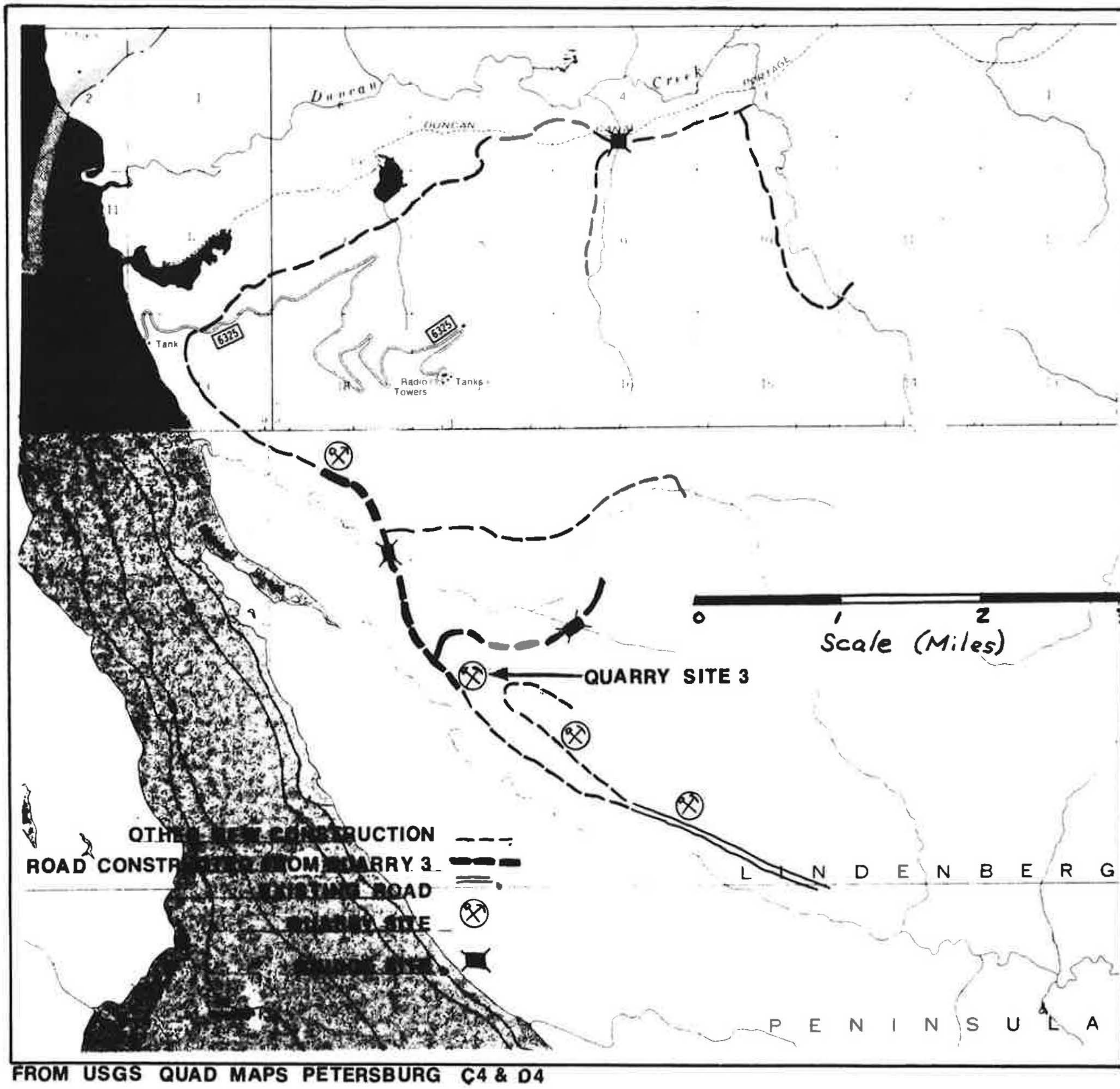

FIGURE 5 Toncan timber sale area.

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.

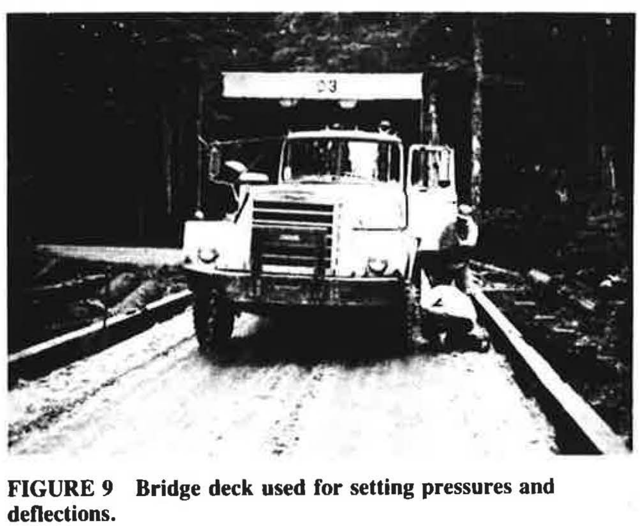

FIGURE 9 Bridge deck used for setting pressures and deflections.

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.

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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.

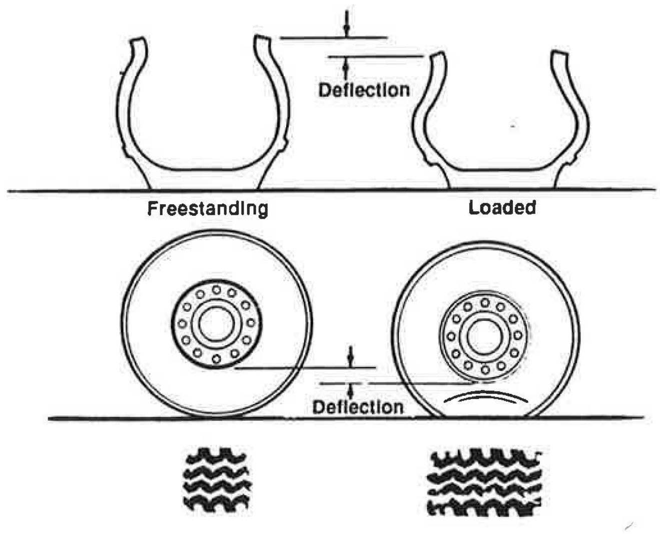

FIGURE 1 Tire deflection.

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 RE0 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. DellaMoretta, 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*).

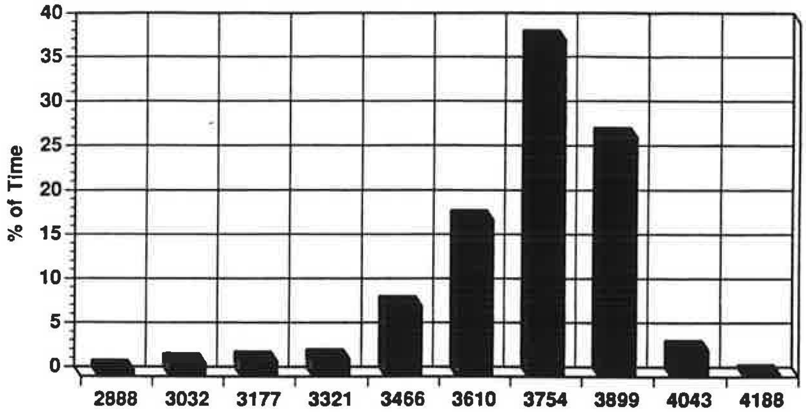

FIGURE 2 Drive shaft torque 10 percent adverse, 10 percent deflection.

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.

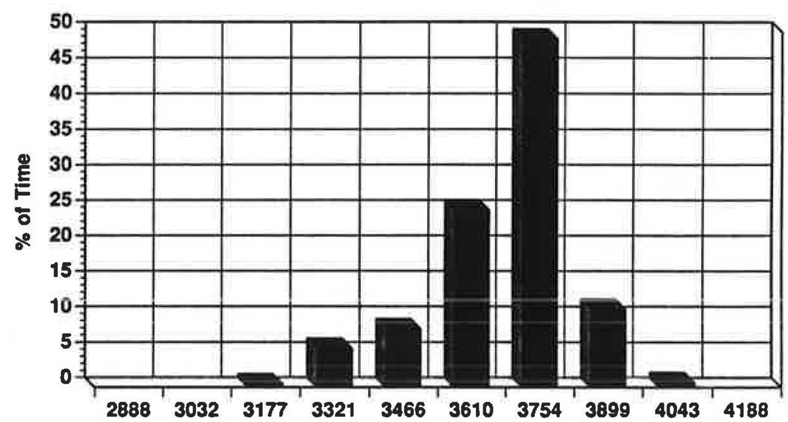

FIGURE 3 Drive shaft torque 10 percent adverse, 15 percent deflection.

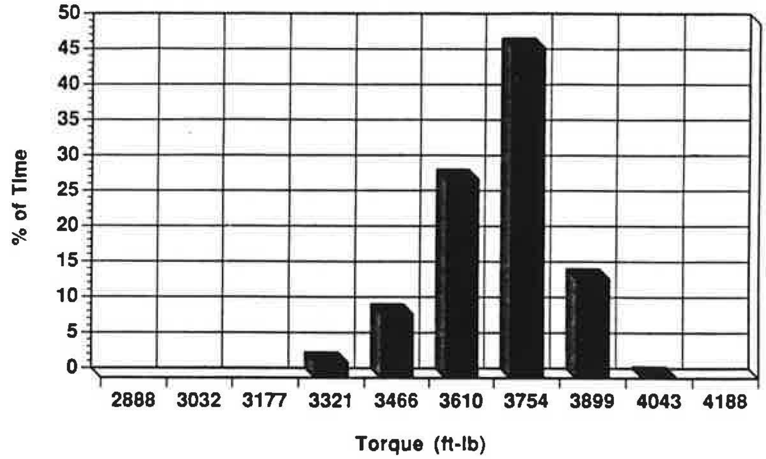

FIGURE 4 Drive shaft torque 20 percent adverse, 20 percent deflection.

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 =

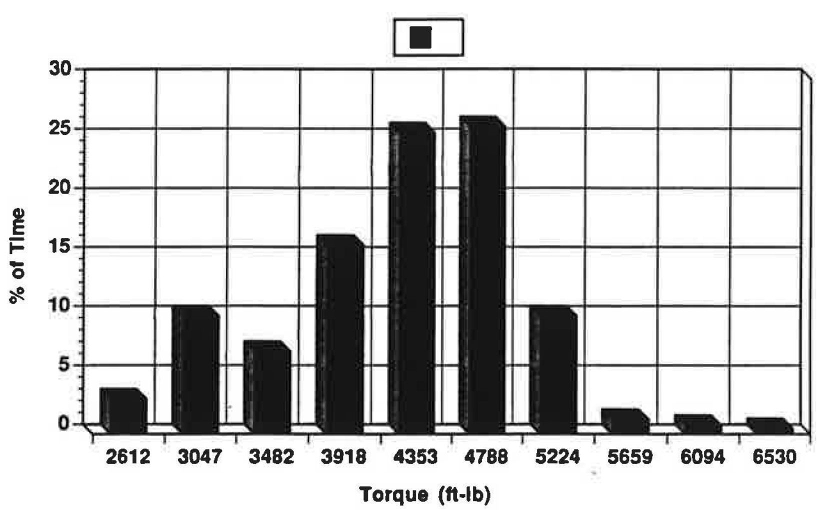

FIGURE 5 Drive shaft torque 25 percent adverse, 10 percent deflection.

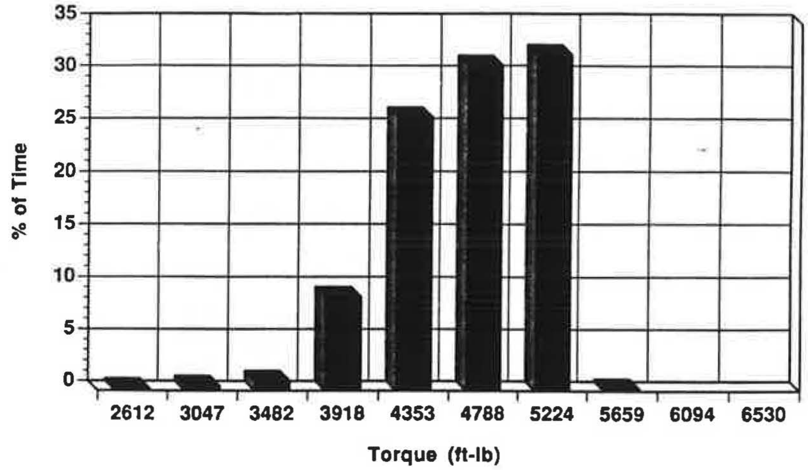

FIGURE 6 Drive shaft torque 25 percent adverse, 15 percent deflection.

(1 − 0.05*N*), 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

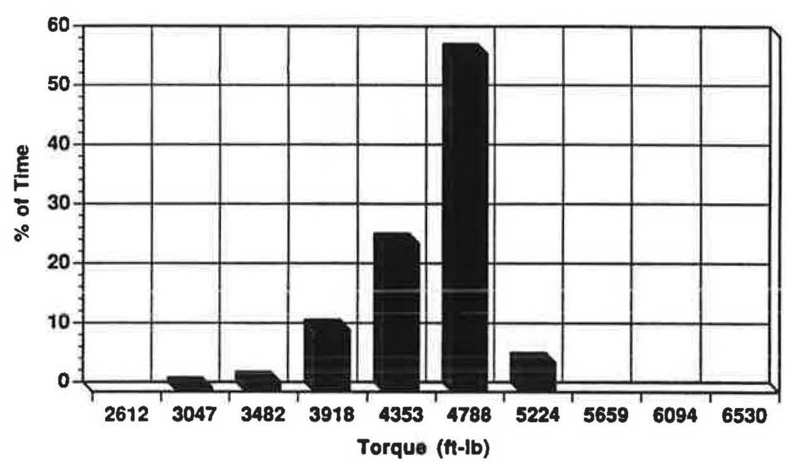

FIGURE 7 Drive shaft torque 25 percent adverse, 20 percent deflection.

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

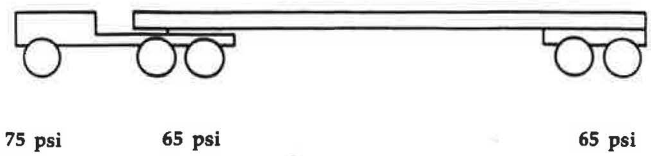

FIGURE 1 Axle configuration and tire pressures for the Quincy and Sonora tests.

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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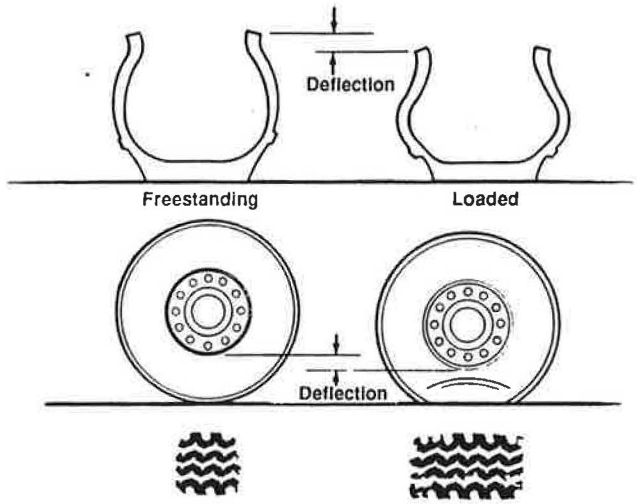


FIGURE 1 Tire deflection.

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:

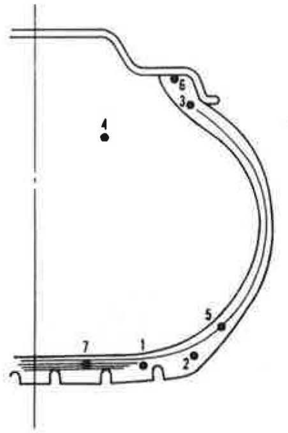
1. Tread wear measurements after 8,833 mi over the test course, which included operation over the severe rock section.
2. Durability determinations of all 36 tires involved in the test. This included both visual observations and X-ray examination of the tire carcass.
3. Tire thermal profile measurements. 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

Tire Damage Noted	*Standard Highway Pressure Vehicle* 8-12% Deflection	*Reduced Pressure Vehicle* 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.

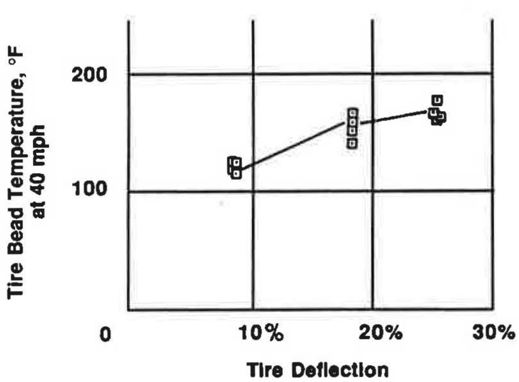


FIGURE 4 Bead temperatures for 11R24.5 drive tires.

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

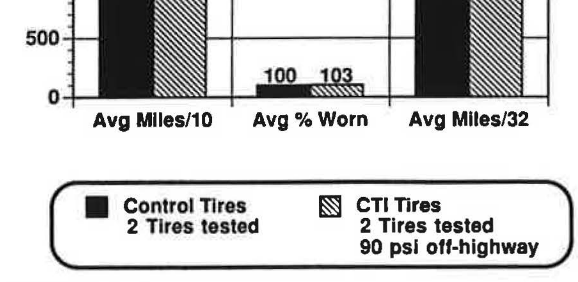


FIGURE 5 Load per tire on steer tires with CTI tires at 90 psi, off-highway (FJM Trucking Co.).

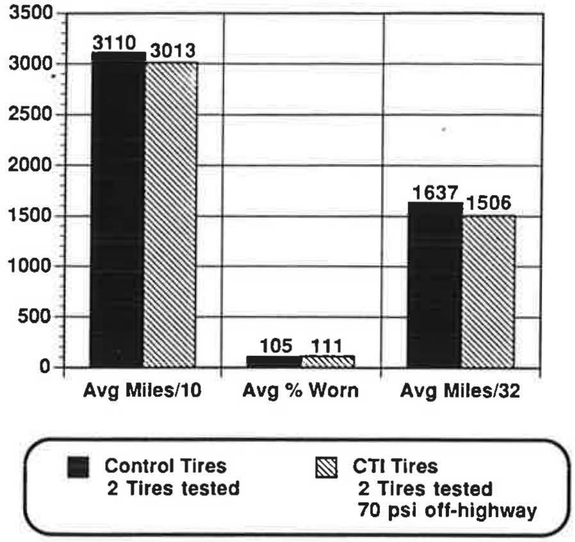


FIGURE 6 Load per tire on steer tires with CTI tires at 70 psi, off-highway (FJM Trucking Co.).

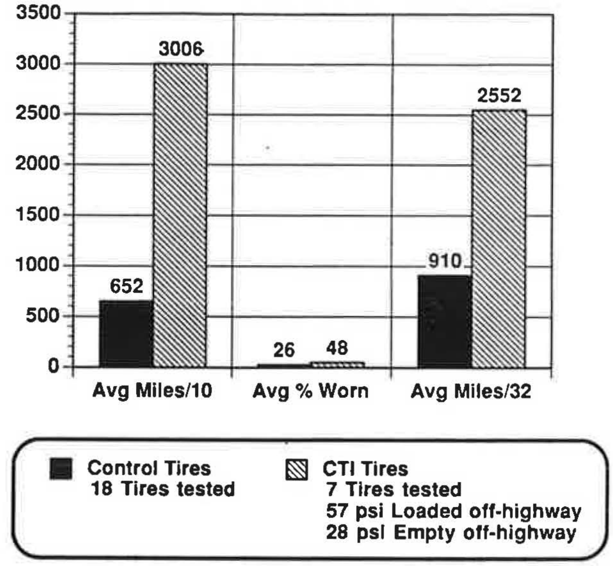


FIGURE 8 Load per tire on drive tires for Retread 1 with CTI tires at 57 psi, loaded, off-highway, and 28 psi, empty, off-highway (FJM Trucking Co.).

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

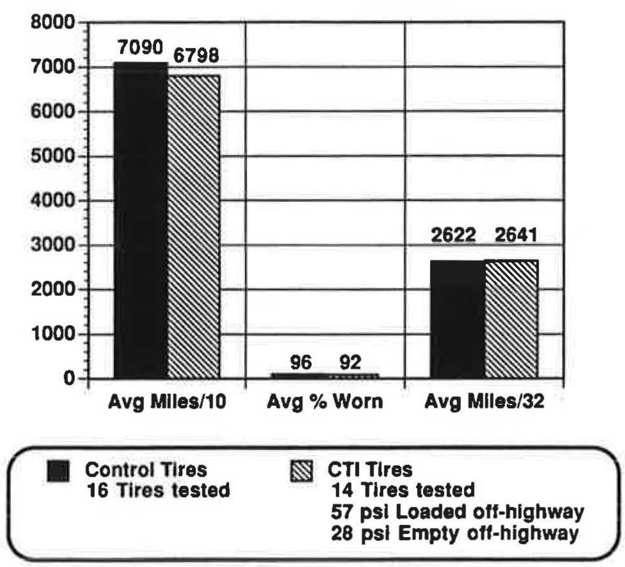


FIGURE 7 Load per tire on drive tires with CTI tires at 57 psi, loaded, off-highway, and 28 psi, empty, off-highway (FJM Trucking Co.).

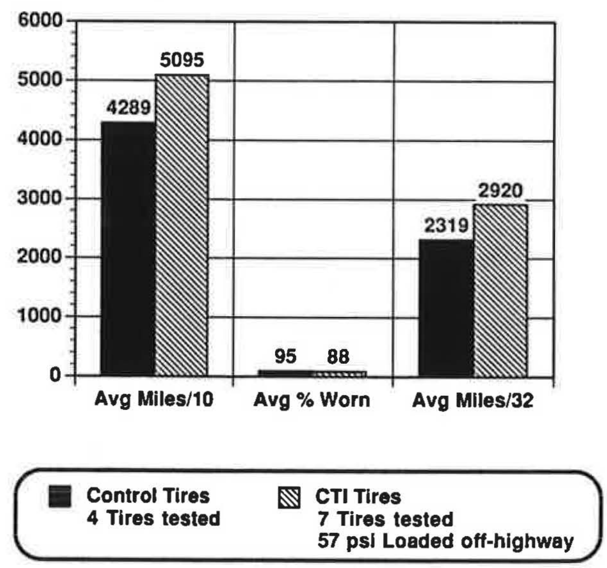


FIGURE 9 Load per tire on trailer tires with CTI tires at 57 psi, loaded, off-highway (FJM Trucking Co.).

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.

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

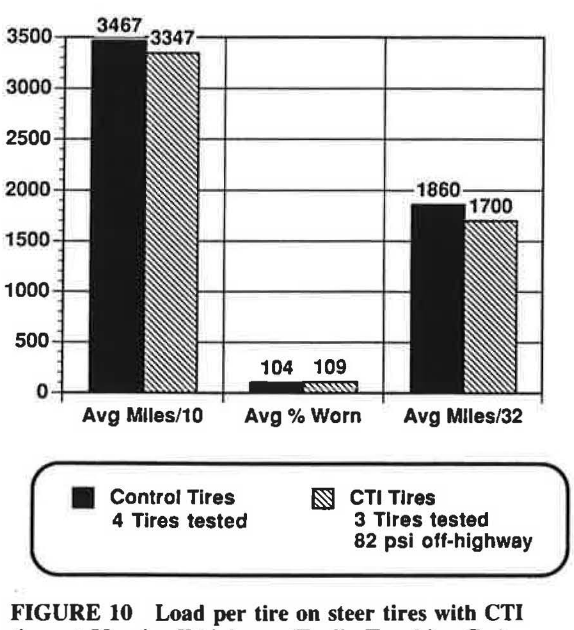


FIGURE 10 Load per tire on steer tires with CTI tires at 82 psi, off-highway (Foglio Trucking Co.).

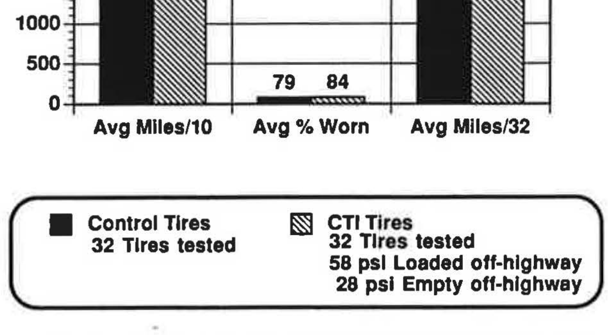


FIGURE 11 Load per tire on drive tires with CTI tires at 58 psi, loaded, off-highway, and 28 psi, empty, off-highway (Foglio Trucking Co.).

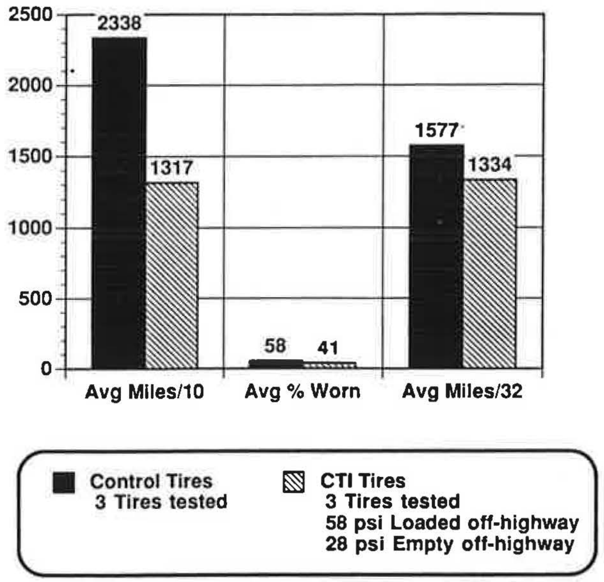


FIGURE 12 Load per tire on drive tires for Retread 1 with CTI tires at 58 psi, loaded, off-highway, and 28 psi, empty, off-highway (Foglio Trucking Co.).

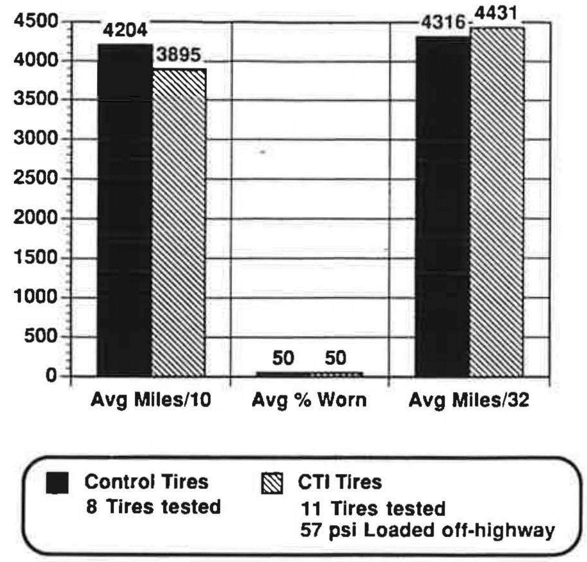


FIGURE 13 Load per tire for trailer tires with CTI tires at 57 psi, loaded, off-highway (Foglio Trucking Co.).

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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