1106

TRANSPORTATION RESEARCH RECORD

Fourth International Conference on Low-Volume Roads

Volume 2

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. 1987

Transportation Research Record 1106
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Printed in the United States of America

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

Fourth International Conference on Low-Volume Roads

(Transportation research record, ISSN 0361-1981 ; 1106)
1. Low-volume roads—Congresses. I. National Research Council (U.S.). Transportation Research Board. II. Series.
TE7.H5 no. 1106 [TE228.5] 380.5 s [625.7] 87-17940
ISBN 0-309-04454-5

Sponsorship of Transportation Research Record 1106

GROUP 5—INTERGROUP RESOURCES AND ISSUES
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The organizational units, officers, and members are as of December 31, 1986.

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Special Report 218, Volume 1

In the keys to Figures 2-4 (page 30) and 3-1 (page 38), two of the definitions were transposed. The middle curve in Figure 2-4 is for male and female drivers, and the bottom curve is for female drivers. In Figure 3-1 the curve represents fatalities per 10,000 population, and the shaded bars represent thousands of fatalities. The correct figures are shown below:

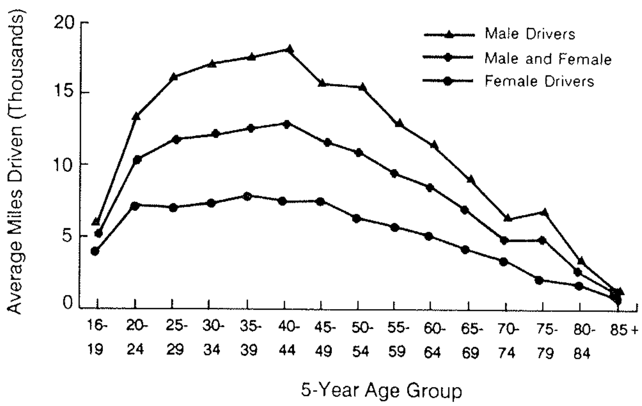

FIGURE 2-4

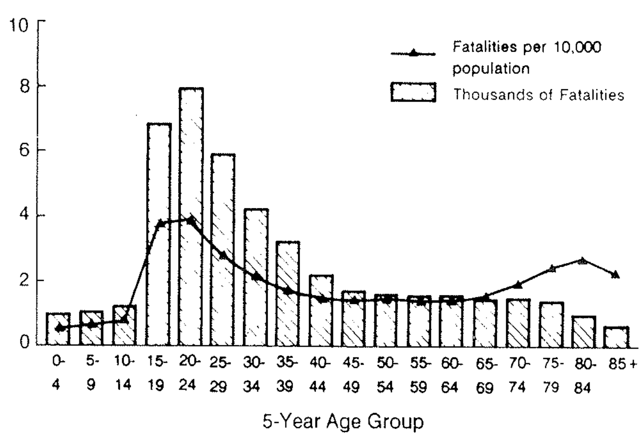

FIGURE 3-1

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page 42
Equation 5 should read as follows:

$$\log_{10}(\varepsilon_{yy})_{ov} = -0.689 + 0.793 \log_{10}(\varepsilon_{yy}) - 0.041(H_{ov} + H_1)^{1/2} - 0.057(H_{ov})$$

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Volume 2

page 225
The second sentence in the abstract should read as follows:

> In the past, a uniform set of geometric design standards for these types of roads was not available in Canada.

page 227
The last item in the bulleted list in column 1 should read as follows:

> One-lane, two-way resource development roads for ADTs up to 100 vpd.

pages 229–232
The figure captions should read as follows:

FIGURE 1 Cross-section elements for two-lane, low-volume earth and gravel roads.
FIGURE 2 Cross-section elements for two-lane, low-volume surfaced roads.
FIGURE 3 Cross-section elements for one-lane, two-way low-volume roads.
FIGURE 4 Cross-section elements for one-lane, one-way low-volume roads.
FIGURE 5 Roadway width versus design speeds of various road agencies (ADT < 50).
FIGURE 6 Roadway width versus design speeds of various road agencies (ADT 50 to 100).
FIGURE 7 Roadway width versus design speeds of various road agencies (ADT 100 to 150).
FIGURE 8 Roadway width versus design speeds of various road agencies (ADT 150 to 200).

Supplement

page 50
In the paper by Faiz and Fossberg, references 3 and 4 were transposed. The last two references in the paper should read as follows:

3. *Road Deterioration in Developing Countries.* Report 6968. Infrastructure Department, World Bank, Washington, D.C., Oct. 15, 1987.
4. M. Mason. *Axle Loading Study.* Internal report. Transportation, Water and Telecommunications Department, World Bank, Washington, D.C., 1981.

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page 30
The last two columns in Table 2 are incorrect. The correct Table 2 is shown at the top of the next page.

TABLE 2 SITES WITH MERGING-RELATED ACCIDENTS

Site No.	Climbing Lane Length and AADT	Vertical Alignment	Horizontal Alignment	Sight Distance	Passing Ahead	Accidents[a] Total and Rate	Merging-Related
1	0.13 mi 3,400	Up 8.5% No crest	Tight curve	Restricted	Very Restricted	20 1,612[b]	1 #1[c]
2	0.73 mi 9,725	Up 6.0% No crest	No curves	Excellent	Restricted	10 282[b]	1 #2[c]
3	0.81 mi 9,725	Up 5.0% Crest	After a curve	Good	Average	12 364[b]	1 #3[c]
4	0.16 mi 11,000	Up 5.9% No crest	Slight curve	Good	Restricted	17 466[b]	5 #4–8[c]
5	0.22 mi 2,200	Up >5% No crest	In middle of tight curve	Restricted	Restricted	6 747[b]	1 #9[c]
6	0.28 mi 2,200	Up >5% No crest	In curve	Restricted	Very Restricted	4 498[b]	2 #10–11[c]

a - Accidents within ±0.10 mile of end of merging taper, 1980-84.

b - Accidents per 10^8 vehicle miles.

c - Numbers refer to accidents described in text.

Source: California Department of Transportation photolog, site plans, correspondence, TASAS, and (6).

page 34

The first sentence in the last paragraph in column 1 should read as follows:

For car speeds of 38 mph, truck speeds of 22 mph, and speeds of other slow vehicles of 26 mph, the following results are obtained.

Transportation Research Record 1131

The paper by Lacy and Pannee (pp. 99–106) was sponsored by the Committee on Engineering Fabrics.

Transportation Research Circular 330

Portions of the original publication were printed in an incorrect sequence. A revised photocopy of the circular is available on request from the Business Office, Transportation Research Board, 2101 Constitution Avenue, N.W., Washington, D.C. 20418 (telephone 202-334-3218).

NCHRP Synthesis of Highway Practice 138

page 4

The data in Table 2 are incorrect. The following table should be used in place of Table 2:

Speed (mph)	Speed (km/h)	Stopping Sight Distance (ft)	Stopping Sight Distance (m)
30	48	200	61
40	64	325	99
50	80	475	145
60	97	650	198
70	113	850	259

NCHRP Synthesis of Highway Practice 139

page 61

In reference 1, the date for "Accident Facts," published by the National Safety Council, should be 1986.

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NCTRP Synthesis of Transit Practice 7

pages 22 and 23
The captions for Figures 11 and 12 have been transposed.

NCHRP Synthesis of Highway Practice 128

page ii
The subject area Bituminous Materials and Mixes (31) is incorrect; it should be Transportation Safety (51). (This is also incorrect in the 1988 TRB Publications Catalog, p. 24.)

NCHRP Report 298

page 93, Section 14.2, Definitions
The last three items in column 1 and the first in column 2 should read as follows:

$$S = \text{Shape factor of one layer of a bearing}$$

$$= \frac{\text{Loaded Area}}{\text{Effective Area Free to Bulge}}$$

$$= \frac{LW}{2h_{ri}\,(L + W)} \quad \text{for rectangular bearings}$$

$$= \frac{D}{4h_{ri}} \quad \text{for circular bearings}$$

page 94
In the third paragraph, last sentence, the reference to "Section 25.9.1" should read "Section 25.7.1."

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Foreword

Low-volume roads constitute the vast majority of the world's streets and highways. In virtually all countries they are the primary means of access to agricultural, mineral, and forest resources as well as to recreational areas. Despite the importance of low-volume roads, researchers, professional practitioners, and administrators have paid relatively little attention to them. As a result, in many countries the lack of maintenance and rehabilitation associated with the low priority accorded these roads has led to deterioration of the low-volume road system.

A workshop on low-volume roads was held in 1975 (*Special Report 160: Low-Volume Roads*). Interest in the design and operation of these roads was such that a second conference was held in 1979 (*Transportation Research Record 702: Low-Volume Roads: Second International Conference*) and a third in 1983 (*Transportation Research Record 898: Low-Volume Roads: Third International Conference*).

During the Third International Conference on Low-Volume Roads the following interrelated high-priority needs were identified:

- More effective methods of correlating and disseminating technical and management information in both developed and developing countries;
- Program management systems to aid in designing, constructing, and managing low-volume roads and to provide information that can be used for justifying appropriate funding levels and establishing appropriate engineering standards;
- Greater understanding of the mechanisms of damage to low-volume roads, particularly the effects of heavy loads on lightly paved or unpaved roads; and
- Characterization of marginal, substandard, or unconventional materials and documentation of their use in low-volume roads.

These four priority concerns were used as the primary basis for planning the Fourth International Conference on Low-Volume Roads held at Cornell University, Ithaca, New York, August 16–20, 1987. The fourth conference was sponsored by the Transportation Research Board Committee on Low-Volume Roads and was organized by a steering committee. Cosponsorship and funding were provided by the Federal Highway Administration, U.S. Department of Transportation, and the Office of Transportation and the Forest Service, U.S. Department of Agriculture. The conference was hosted by Cornell University and the Cornell Local Roads Program with the assistance of the New York State Department of Transportation.

Representatives from countries around the world participated in the Fourth International Conference on Low-Volume Roads at which the importance of technology transfer was the subject of the keynote address by John Metcalf and of a special conference session. A majority of the 77 papers in these two volumes are on the results of technical research on design, construction, and maintenance of low-volume roads.

The technical needs cited by the participants in the third conference were addressed in the fourth conference; the use of asphalt seals and concrete in rural roads and the design and construction of low water crossings and bridges were also discussed.

Although technical problems are being addressed with increased vigor, there are relatively few international organizations, and even fewer national agencies, that are developing and disseminating in-depth knowledge of the social and economic, as well as the technical, aspects of the world's low-volume roads. The Fourth International Conference on Low-Volume Roads afforded an opportunity for interested persons from around the world to share information, and this two-volume Record, which contains preprints of the papers presented at the conference, represents a significant contribution to knowledge about low-volume roads.

The TRB Committee on Low-Volume Roads recognizes and appreciates the contribution of the following members of the Steering Committee for the Fourth International Conference on Low-Volume Roads:
Mathew J. Betz, Arizona State University, chairman;
Gary R. Allen, Virginia Highway and Transportation Research Council;
Asif Faiz, The World Bank;
Lynne H. Irwin, Cornell University;
Richard Lanigan, Delaware County, New York;
Melvin B. Larsen, Illinois Concrete Council;
Ruth McWilliams, Office of Transportation, U.S. Department of Agriculture;
John Pruitt, Forest Service, U.S. Department of Agriculture;
Richard Robinson, Transport and Road Research Laboratory, United Kingdom; and
A. R. Van De Meulebroecke, Federal Highway Administration, U.S. Department of Transportation.

The contributions of TRB staff members Neil F. Hawks and George W. Ring III are also hereby acknowledged.

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An Overview of Alternate Surfacings for Forest Roads

MOJTABA B. TAKALLOU, R. D. LAYTON, R. G. HICKS, AND JOHN LUND

The use of earth or aggregate surfaces for forest roads has persisted as a result of the low initial cost of crushed aggregates. In recent years, the shortage of crushed aggregates and the long hauls associated with their use have resulted in higher costs for quality aggregates in many parts of the country. The possible use of more economical, unique materials for surfacing forest roads is investigated. A discussion is presented of the necessary background information needed to identify and evaluate the alternate surfacing systems. Guidelines are provided of the applicability of each surfacing system for specific areas and situations. The alternate systems considered include those that are (a) capable of being moved as the hauling or mining activity moves, (b) low-cost materials compared to good-quality aggregates, (c) marginal materials that have relatively short lives but satisfy the project life, and (d) materials that are available in the desired areas to reduce construction costs. A comprehensive market review and literature search, and the construction of demonstration projects were undertaken to evaluate various alternate materials for surfacing forest roads. Potential surfacing types and methods included biodegradable materials (wood and bark chips), chemical stabilization, geotextiles, marginal aggregates, sand-sealed subgrade, metal mats, reusable aggregates, a membrane-encapsulated soil layer, and geoweb stabilization. The results of this research indicate that several of these alternate surfacings can perform satisfactorily and be cost-effective. The most viable alternates include a) wood and bark chips, b) chemical stabilization, c) marginal aggregates, d) reusable aggregates with or without inexpensive geotextiles, e) a sand-sealed membrane, and f) steel mats for emergency situations and short projects.

Of the nation's 3.88 million miles of roads and streets, 1.86 million miles are either unsurfaced or are surfaced only with stone, slag, or gravel (*1*). An additional 1.0 million miles have a minimum surfacing, ranging from surface treatments and chip seals to no more than a 7 in aggregate surface. The U.S. Forest Service operates one of the largest low-volume road networks under the jurisdiction of a single agency in the world. This system contains approximately 330,000 miles of roads of which 92,400 miles have an aggregate surface and 221,000 miles are unsurfaced (*2, 3*). The agency continues to construct and reconstruct 11,000 miles of road annually with an annual expenditure for construction, reconstruction, and maintenance of over $500 million (*2*). Log trucks are one of the major users of these roads. These heavy trucks produce high stresses in the road surfacings; however, the number of repetitions are relatively small. About 90 percent of the roads constructed by the Forest Service are constructed solely for logging traffic that might last a few seasons (or only one) and carry less than 100 vehicles per day (vpd) (*2*).

High-quality aggregate is often not available near the project site, which results in high transport costs. Consequently, the possible use of innovative technology and unique materials for road surfaces was investigated to find more economical surfacing materials. Some of these surfaces could serve a temporary or intermittent use in which the surfacing can be removed and reused again, such as aluminum, steel, and geotextile mats. Other surfaces could be improved by means of soil stabilization, a membrane-enveloped soil layer (MESL), and expandable grids. Finally, some of the surfaces may be economical because of the ready availability of the materials in the desired regions, such as wood, bark chips, and marginal aggregates.

The results are presented of a study sponsored by the USDA Forest Service to identify and evaluate the feasibility of a variety of alternate surfacing systems for use on temporary or intermittent-use roads. The results of a literature study and a field evaluation of 11 demonstration projects are specifically presented in this paper.

POTENTIAL ALTERNATE SURFACING SYSTEMS

A comprehensive literature review and field data collection effort were undertaken to identify various materials and their properties that could be suitable for surfacing temporary and intermittent-use roads. This effort provided the background needed to select the most viable alternate surfacings for use in the field evaluation phase.

Criteria for Alternate Surfacings

The decision of when, where, and under what conditions alternate surfacings should be used is based on traffic conditions, road objectives and needs, subgrade type, materials characteristics, and, most importantly, economics. The following criteria were considered important for defining materials and situations that would be effective applications of alternate surfacings.

Roadway Characteristics

- Temporary or intermittent-use roads,
- Volumes of traffic of 50 to 100 vpd,
- A required surfacing life of 1 to 5 years,
- Short hauling distances from one project to another,
- Poor, low-strength subgrades, and
- Inaccessible or long haul distances.

M. B. Takallou, University of Portland, Portland, Oreg. 97203. R. D. Layton and R. G. Hicks, Oregon State University, Corvallis, Oreg. 97331. J. Lund, Oregon Institute of Technology, Klamath Falls, Oreg. 97601.

Material Characteristics

- Limited or severely restricted supply of good-quality aggregates,
- Materials capable of being moved as logging moves,
- Low-cost materials compared to good-quality materials, and
- Materials available in the project area.

Description of Potential Surfacing Types

A wide variety of alternate surfacing systems were reviewed and investigated to determine their practicality and cost-effectiveness for use on temporary or intermittent-use low-volume roads. The criteria just described were applied to select those materials that had the greatest potential of being effective. A summary of the materials found to have a strong potential for the characteristics of cost-effectiveness, adequate strength, availability, and environmental acceptability is given in Table 1. Each of these surfacing systems is described below.

Biodegradable Materials

Biodegradable materials, including bark and wood chips, sawdust, and planks or logs, have previously been used as road surfacings (*4*). The cost of the wood chip materials ranges from about \$3.00 to \$10.00/yd^3 in-place or \$0.17 to \$0.55/ft^2 for a 12-in surface layer. The wood and bark chips used to construct these roads typically have a size no larger than 6 in. Chips are generally hauled in dump trucks or chipped on site. Compaction from construction equipment is generally sufficient.

Soil Stabilization

Soil stabilization as a technology has been in use for over 50 years. Numerous different stabilizing agents have potential for use in temporary and intermittent-use roads. Lime, lime-fly ash, cement, asphalt emulsions, sodium chloride, magnesium chloride, calcium chloride, and lignin sulfonate have all been used as surfacing systems for low-volume roads. The nature of the subgrade soil is of particular importance in dictating which stabilizing agent should be used.

Marginal (or Degradable) Materials

Marginal materials are those that nearly meet standard American Association of State Highway and Transportation Officials (AASHTO), American Society for Testing and Materials (ASTM), or agency specifications for road use. They may require some type of additive or special treatment to perform in the environment in which they are to be used (*5*). Many types of marginal aggregates can be found around the country that can provide satisfactory performance if upgraded. These include, but are not limited to, materials such as cinders, pumice, decomposed sandstone, marginal sand, sand-clay, chert, basalt, and topsoil.

Sand-Sealed Subgrade

The application of an emulsified asphalt on a natural subgrade followed by the application of sand as a topping to waterproof the subgrade soil is called a sand-sealed subgrade (*6*). The sand-sealed subgrade has been used in areas that have a good

TABLE 1 CHARACTERISTICS OF ALTERNATE SURFACING SYSTEMS

Type of Material	General Description	Material Cost	Expected Life
Biodegradable materials (wood or bark chip)	12-24 in of wood or bark chips	\$3-7/yd^3 or \$0.11-0.25/ft^2 [a]	1-3 years
Chemical stabilization	Lime, lime fly ash, portland cement, asphalt emulsion, NaCl, $CaCl_2$, MgCl, lignin sulfonate	\$0.25-0.45/ft^2 [b] (varies greatly with locality)	5-10 years
Geotextile and Geogrid separation	Tensar grids (SSI), various fabrics under crushed rock	\$0.05-0.40/ft^2 plus aggregate	1-3 years
Marginal Aggregate	Single or double layer of sand sealed with CRS-2	\$0.15-0.25/ft^2	3-5 years
Metal mats	Aluminum (AM-2) or steel (M8A1) mats	\$8.33/ft^2 and \$0.90/ft^2 respectively	5-10 years
Reusable aggregate without geotextile separation	6-18 in of crushed aggregate on subgrade	\$0.05-1.10/ft^2 [a] (construct) \$0.20-1.30/ft^2 (recover)	5-10 years
Reusable aggregate with geotextile separation	Fabrics on subgrade with 6-18 in crushed aggregate	\$1.0-1.50/ft^2 [a] (construct) \$.50-1.00/ft^2 (recover)	5-10 years
Membrane encapsulated soil layer	6-24 in subgrade soil encapsulated with various membranes	\$0.50-1.30/ft^2	3-5 years
Geoweb stabilization (expandable grids)	8 in dune sand filled plastic grids, sealed with asphalt	\$1.05-1.30/ft^2 \$1.5-2.0/ft^2 in place	5-10 years

[a]Assumes 12 in surface thickness
[b]Assumes 5% stabilization agent and 6 in depth of stabilized layer

and firm subgrade (CBR greater than 15; R value greater than 40) and that have a plentiful source of low-cost sand compared to crushed aggregate (*6*).

Prefabricated Mat Panels

Landing mats are surface-covering panels that are prefabricated from materials such as aluminum, steel, and fiberglass. The mats come in various panel sizes and can be connected to form a continuous road surfacing. Mats may be effective when temporary surfacings are needed to carry large and heavy trucks or equipment. As a result, they may be applicable for temporary and intermittent-use roads because they can be reused, stored, installed, and removed. Steel mats are not currently being produced but are sometimes available through acquisition of U.S. government surplus equipment (*7*).

Reusable or Recycled Aggregates

The recovery and reuse of good-quality aggregates with or without geotextile as a separation layer may offer an acceptable solution to the shortage of aggregates and high-cost, quality aggregates for temporary or intermittent-use roads. The use of good-quality aggregates with geotextiles and reuse of one or both materials in several projects must account for the properties of materials, availability of materials, performance, and economics.

Membrane-Encapsulated Soil Layer (MESL)

The MESL concept is a method for maintaining the moisture content of the soil at a desired level by encapsulating the soil in a waterproof membrane (*8*). The Waterways Experiment Station developed a method for MESL that consists of first excavating and stockpiling fine-grained soils. Its moisture content is adjusted to 2 to 3 percent below the optimum moisture content for the specified compaction. Following compaction, a CRS-2 asphalt emulsion is sprayed on the subgrade to hold the membrane in place. The excavated soil is replaced and compacted to the desired density and moisture content. An asphalt emulsion is sprayed on the surface and a surface membrane is installed. The top membrane is also sprayed with an asphalt emulsion and covered with a thin layer of clean sand to blot the asphalt and to provide added protection against puncture by construction equipment (*8-10*). Membrane materials used include polyethylene, rubber, vinyl, and polyesters.

Grid Confinement

Grid confinement is a concept for pavement base course construction that was developed at the U.S. Army Corps of Engineers Waterways Experiment Station (*7*). The concept involves the confinement of sand in interconnected cellular elements, called grids, to produce a load-distributing pavement base layer. Poorly graded sands, generally found around the world, can be used in expedient construction of sand-grid base layers for many pavement applications. Three types of grids are available. These include paper grids, aluminum grids, and, most encouragingly, plastic grids (*7, 11*). The plastic grid, called a geoweb, is composed of strips of high-density polypropylene sheets that are spot-welded together on 13-in (33.02-cm) centers (see Figure 1). When the system is expanded, it opens like an egg crate divider into an 8-ft (2.48-m) $\times$ 20-ft (6.10-m) panel. The panels are expanded, set in place, and filled with sand. The grids are then compacted and the surface is sprayed with a liquid asphalt. The grid materials currently cost about \$1.25/ft^2 of expanded area and the construction costs range from 30 to 60 cents/ft^2 (*11*).

DEMONSTRATION PROJECTS

Eleven demonstration projects were studied to evaluate the construction qualities and performance of the alternate surfacing materials. Six projects were constructed in the summer of 1984. The other five projects had already been constructed. Although most of the projects were in the Pacific Northwest, some were located in the Southeast and Southwest.

Evaluation of the demonstration projects included monitoring the construction, performance, maintenance, and, if necessary, recovery operations. The characteristics of the already existing and new demonstration projects are summarized in Tables 2 and 3.

Demonstration Project Plans

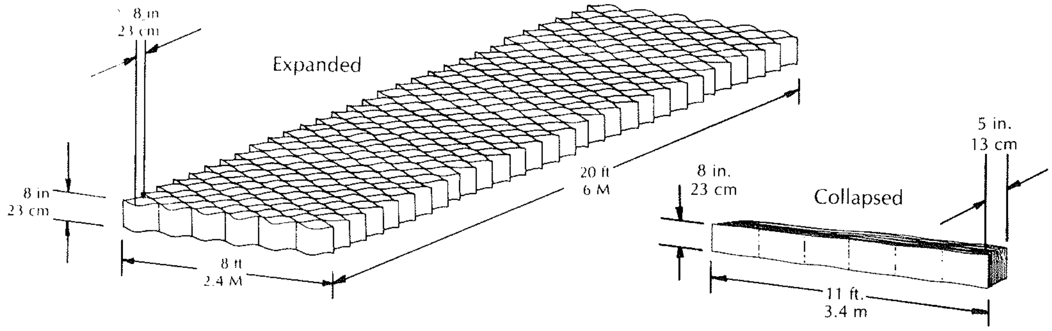
Construction was undertaken with Forest Service maintenance crews for most projects. Materials were obtained through purchase orders. Information collected for both projects included such data as location, general project description, construction requirements, removal and reuse, and estimated costs.

Information collected for project monitoring purposes on each of the projects consisted of a) environmental and traffic data, b) design and construction data, c) maintenance data, d) cost data, and e) performance data. Most of these data were collected using standardized project data sheets. Some of the information had to be estimated, based on interviews with vehicle operators, construction workers, and maintenance personnel. This included qualitative information on the ability of each pavement type to be constructed, to be maintained, and to handle traffic loads.

Results

Wood and Bark Chip Project

This road surfacing consisted of 6-in minus wood chips. Two roads were constructed in late 1981 and surfaced with up to 24 in of chips. Five other roads in the same area were surfaced with aggregate for comparison. The subgrade soil consisted of a clayey silt to sandy silt with an SMu and ML classification, respectively. Prior to construction, 87 percent of the wood chips passed the 6-in sieve and 12 percent passed the No. 4 sieve. Two years after construction, 100 percent passed the 3-in sieve and approximately 84 percent passed the No. 4 sieve. Although the surface was expected to last only for the duration of the timber sale, it was still in good condition as of May 1984. As of May 1984, the ruts were about 2 to 3 in deep. However, traffic was



GEOWEB Structural Properties	English System	Metric System
1. Expanded Dimension	8 ft. x 20 ft. x 8 in.	2.5 m x 6 m x 20 cm
2. Collapsed Dimension	11 ft. x 5 in. x 8 in.	3.4 m x 13 cm x 20 cm
3. Panel Thickness Nominal	0.047 in.	0.119 cm
4. Weight	5.7 lb./yd^2	3.1 kg/m^2
5. Cell Area	41 in.^{2}	265 cm^2
6. Cell Seam Node Pitch	13 in.	33 cm
7. Welds/Seam	7	7
8. Seams Tensile Peel Strength	150 lbs.	69 kg
9. Installation Temperature Range	-16°F to 110°F	-27°C to 43°C

Polymer Material : High Density Polyethylene
Color : Black
Carbon Black Content: 2%
Chemical Resistance : Superior

FIGURE 1 Geoweb confinement system.

light; it consisted of approximately 25 to 30 log trucks and some recreational and administrative traffic. No maintenance was required.

The total cost of construction for two road sections was estimated to be an average of \$0.74/ft^2 for wood chips and \$0.99/ft^2 for crushed aggregate. The wood chips cost approximately \$5.00/yd^3 whereas the aggregate cost about \$15.00/yd^3.

Chemical Stabilization Projects

Several road projects in Region 8 (Southeast) were constructed by mixing lime, cement, bottom ash, or pozzolime with native soils. In some instances, the stabilized soil was used as the wearing surface. In other cases, a thin gravel surface was used.

The surface of the pozzolime-modified soil projects was slippery after rainfall with both sand and clay subgrades, which resulted in a failure of bearing capacity. The pozzolime did not react well with the sand and clay soils initially but gained strength after an additional year of curing and is currently in good condition. On another project, the clay subgrade soil of a road modified with pozzolime and bottom ash performed fairly well until logging operations disrupted drainage and caused saturation of the road bed. A failure in bearing capacity resulted. The road is currently in fair condition. Another road, which consisted of a lime-modified clay subgrade with a gravel surface, developed slick conditions, some potholing, deep rutting, and bearing capacity failures after freeze-thaw conditions. Construction problems may have produced insufficient strength for successful soil modification to take place.

Despite these problems, there were also many successes. For example, a road of pozzolime-modified clay with a gravel surface constructed for a design traffic volume of 7,000 18-kip equivalent axle loads is in excellent condition. The clay soil of another road with a gravel surface was modified with a combination of pozzolime and fly ash and Portland cement and fly ash. The road was built to carry a traffic volume of 2,000 18-kip equivalent axle loads and is also in excellent condition. Finally, a road constructed of Class C fly ash-modified shale gravel base and a surface course that is designed to carry 2,000 18-kip single-axle loads is currently in excellent condition.

The costs ranged from \$0.56/yd^2 to \$4.31/yd^2 for a typical 6-in stabilized layer with an average cost of \$1.74/yd^2, compared to \$2.29/yd^2 for quality aggregate roads.

TABLE 2 EXISTING DEMONSTRATION PROJECTS — SUMMARY TABLE

Project No., Title, Location	Subgrade Soil Type	Surface Type	Cost
Wood and bark chip: Mt. Baker-Snoqualmie NF, WA Region 6 (1981)	Clayey silt to sandy silt SMu to ML	12, 18, and 24 in wood chips	$0.74/ft^2 [a] avg ($5/yd^3)
Chemical Stabilization: Region 8 (1978-1984)	GM, SW, SP, and CH	4-8 in modified subgrade using cement, pozzolime, or fly ash	$0.20/ft^2 [b]
Geotextile and geogrid separation:[a] Olympic,[b] Siuslaw, and [c] Willamette NF Region 6 (1976, 1983, and 1983)	1. Organic clay and OH-MH 2. Gravelly and sandy SMu 3. Silt-MH	Gravel	$.05-.40/ft^2 aggregate
Substandard Aggregate: Siuslaw and Willamette NF, OR, Region 6 (1983)	Clayey silt (ML) and sandy silt (SMd)	4-10 in of substandard pit run and crushed rock	$1.15-4.70/yd^3 (or $.04-.17/ft^2) [a]
Sand Seal: Plumas NF, CA Region 5 (1975)	Clayey silty sand to rocky clay (ML to SM)	Double sand seal	$0.75/ft^2

[a]Assumes 12 in surface thickness
[b]Assumes 5 percent stabilizing agent and 6 in depth of stabilized layer

TABLE 3 NEW DEMONSTRATION PROJECTS — SUMMARY

Project No., Title, Location	Subgrade Soil Type	Surface Type	Cost
Geoweb stabilized road: Dunes National Recreational Area - Siuslaw NF, OR Region 6 (1984)	Dune sand (SP)	Sand filled plastic grid 8 in deep	$1.84/ft^2
Wood chip roads: Siuslaw NF, OR Region 6 (1984)	Silt to fine sandy silt (ML)	12 in of alder wood chips	$11.17/yd^3 or $0.41/ft^2 [a]
Reused/recycled aggregates: Siuslaw NF, OR Region 6 (1984)	Silt (ML)	Four different fabrics with 8-10 in crushed aggregate	$1.30/ft^2 [a] to construct and $1.22/ft^2 [a] for recovery
Metal mat surfaces: Willamette NF, OR Region 6 (1984)	(ML)	Steel and aluminum mats	$8.70/ft^2 materials and placement, $0.40/ft^2 recovery
Membrane encapsulated soil Layer: Tahoe NF, CA Region 5 (1984)	Volcanic ash and mud flow mixed with glacial till	6 in subgrade soil encapsulated with membranes with chip seal or 3 in gravel	$0.50-1.28/ft^2
Lignin sulfonate soil stabilization: Prescott NF, AZ Region 3 (1984)	Clayey sand (SC) to sandy gravel (GW) - from decomposed granite	4 in of stabilized subgrade covered with a chip seal	$0.25/ft^2 [b] or $0.68/yd^3

[a]Assumes 12 in surface thickness.
[b]Assumes 1.3 percent sulfonate and 4 in depth of stabilized layer.

Geotextile and Geogrid Separation Projects

Projects in three national forests in which geotextiles or geogrids were placed to separate the surface material from the subgrade were investigated. The subgrade soils all had extremely low strengths, which therefore precluded the use of conventional construction practices. These special materials were used to either bridge these weak areas or to reduce the total thickness of the structural section, or both.

One project near Quinault, Washington, was constructed in 1976 using eight different geotextile types and grades (*12*). The subgrade soils were of highly organic clays and silts (Unified classification OH to MH) with a low bearing strength (CBR less than 1). After 2 years of service, the subgrade soil had increased in strength (vane shear) by approximately 250 percent, the moisture content reduced 75 percent (from 218 to 55 percent), and the unit dry weight increased from 78 to 101 pcf. As a result of this gain in strength, the depth of crushed aggregate could theoretically have been reduced from 20 to 10 in. The geotextiles had, on the average, reduced in strip tensile strength to 80 percent of their original values. The average rut depth varied from 0 to 0.37 ft (an average of 0.14 ft), whereas the depths varied from 0.25 to 0.5 ft on the control sections with no fabric. In 1984 the road was in overall good condition; the structural adequacy, and surface and traffic serviceability were at high levels.

The second project was also located in the Northwest. The subgrade soils consisted of gravelly and sandy silts with a classification of SMu and a CBR ranging from 1 to 5. The road was initially constructed in 1980 with 4-oz/yd^2 Mirafi 500X fabric. The performance and maintenance of the project were impaired in many places because rutting developed during construction prior to compaction. The first failures occurred between 100 and 300 18-kip equivalent axle loads; ruts greater than 4 in developed. The road was reconstructed in 1983 with a Tensar (SS-1) geogrid subgrade reinforcement.

In the third project, geogrids were used in 9.8-ft widths. Crushed aggregate was then placed over the fabric to an average depth of 12 in that varied from 9 to 17 in. The strength of the subgrade soil increased with time as a result of consolidation by traffic. Within a year, the strength had increased to an average value of 2,240 psf (vane shear), compared to 1,170 psf at the time of construction. Rutting occurred to an average depth of 3 to 4 in; short segments rutted as much as 12 in.

The evaluation of these projects indicated that the use of geotextile separation or reinforcement appears to be an especially good alternative in cases in which the underlying subgrade is of a poor quality or is highly plastic.

Marginal Aggregates

Demonstration projects in two national forests permitted the use of marginal aggregates as a surfacing material to be evaluated. These aggregates, which did not meet such Forest Service specifications as gradation and durability, were placed directly on the subgrade and were expected to be serviceable for 2 to 3 years. Low-quality, crushed aggregates of pit-run and various gradings were used to construct these roads. The aggregates were placed directly on the subgrade soil. The subgrade soil for one project consisted of a clayey silt (ML) or a silty sand (SMd) material with a CBR at 95 percent of T 99 compaction that ranged from 8 to 19. The other project subgrade soil was a clayey silt (ML) with a CBR of 6. A third project had subgrade soils that ranged from silty sand (SMu) to clayey silt (ML) with a CBR around 7. All projects experienced rutting and potholing, including those that did not carry logging traffic.

Comparative construction costs are as follows:

	Cost/Yd^3		
Material Type	*Project 1*	*Project 2*	*Project 3*
Marginal crushed rock	\$17.00-19.00	\$22.55	\$6.58
Pit-run	—	\$4.69	\$1.15-2.50
Quality crushed aggregate	\$24.50-26.00	\$25.00-28.00	\$11.00-13.00

The economic advantages were reduced or eliminated by the added maintenance costs when the marginal rock was only slightly less expensive than quality crushed rock.

Sand-Seal Project

This demonstration project involved the construction of a sand-seal project and two reconstruction projects. A sand seal was placed directly on the compacted subgrade, which eliminated the need for a prepared subbase or base material. The subgrade soil consisted of a clayey, silty sand to rocky clay from deeply weathered diorite, metagabbro, and serpentine rock. The Unified classification varied from ML to SM; R values ranged from the 20s to 60s, and the CBR ranged from 11 to 19. After about 3 years, half of the surface had failed as a result of either cracking and breaking of the surface or rutting in soft spots. Bleeding of the surface was also a major problem, especially when clean sand was used. Decomposed granite appeared to have enough fines to act as a blotter.

Geoweb-Stabilized Road

This demonstration project tested the use of a three-dimensional plastic grid. Three main problems were identified during construction: (a) an uneven surface as a result of the subgrade preparation, (b) surface leveling of the sand over the top of the grids, and (c) inadequate penetration of the asphalt emulsion. These problems resulted in a lack of bonding of the sand in the top of the grid. Because the loose, unbonded sand on the surface was too deep (2 to 3 in), it failed almost immediately. Traffic was soon avoiding the test section. Because of heavy traffic demands, part of the grid section was torn out and replaced with crushed rock. The loose sand was removed, and 2 to 4 in of crushed rock was placed over the grids on the rest of the section.

The costs of geoweb-stabilized roadways are very high. The cost for this system was \$11.25/yd^2 for the grids and \$5.35/yd^2 for construction and the asphalt emulsion. With experience, the cost could be reduced by approximately 25 percent.

Wood Chip Project

This demonstration project was constructed with 12 in of wood chips that were produced on-site and placed directly over the subgrade for use as a base and surface course. The road section was 200 ft long, the maximum grade was 5 percent, and the expected logging traffic was 1.2 million board feet of timber, or approximately 240 log truck trips. The surfacing material consisted of alder wood chips no larger than 6 in; 100 percent passed a 2-in sieve. The optimum moisture content was around 20 percent, and the maximum density was slightly over 21 pcf. The subgrade soil consisted of silt to fine sand silt (ML classification) with a maximum density of 85.2 pcf and a CBR at 95 percent (ASTM D 698) of 10.0. The cost was \$3.72/yd^2 for the wood chips in the 12-in thick surface.

The Forest Service expected the road to last only for the duration of the timber sale. The only traffic over the roadway has been construction traffic and traffic associated with logging. Speeds over the roadway were generally less than 5 mph. Limited maintenance has been required to repair minor rutting in the surface.

Reusable and Recycled Aggregates

A project to test a reusable and recycled aggregate was undertaken. The project was placed over an ML classification subgrade with a CBR of approximately 5. Four different types and weights of fabric were used to separate the subgrade for the surfacing aggregate, including Exxon GTF-400E, Mirafi 600X, Mirafi HP1200, and Fibretex 400R. Between 8 and 10 in of crushed aggregate were placed on top of the fabric as a base and wearing surface. The aggregate had a USFS grading of "G"; 100 percent passed the 2-in sieve and 4.3 percent passed the No. 200 sieve (*13*).

Aggregate from the project was recovered in late August of 1984. The aggregate was first scraped using a JD 450 bulldozer to within 3 in of the geotextile surface. The recovery operation was extremely difficult and slow because of the rutting and uneven subgrade. In some cases, the bulldozer damaged the geotextile in spots in which the aggregate thickness was only 1 in. The edge of the geotextile was then exposed using shovels. A recovery beam was then sewn to 3 ft of the edge of the exposed fabric. Cables with turnbuckles were attached to the beam, and the other end was attached to the front-end loader (see Figure 2). Finally, the fabric was pulled back by the loader and cables, rolling the aggregate into a cylindrical pile. The aggregate pile was then loaded into trucks and the fabric was salvaged. Some tearing of the fabric occurred in all sections.

Construction of the section cost about $11.70/yd^2 and 90 percent of the 277 tons of aggregate placed were recovered and reused. It is expected that the production rate could have been nearly doubled if only the aggregate was recovered. The net savings associated with the recovery operation as a result of the value of the recovered materials is about $5.67/yd^2. Thus, the net recovery cost was about $6.03/yd^2. The project probably would have been more economical if an inexpensive fabric had been used and only the aggregate had been recovered.

FIGURE 2 Attaching and pulling the recovery system.

Metal Mat Surfaces

This project tested the use of metal mats as a wearing surface placed directly on the subgrade. The project was constructed in 1984 and was surfaced with both steel and aluminum mats that were 12 ft wide. No special problems were encountered when the mats were laid other than the occasional need to bend some of the tongues on the steel mats so that they aligned properly.

Because the steel mats are no longer manufactured, only the 1960 cost of $0.90/ft^2 is available. Current 1987 costs for the aluminum mats are $9.18/ft^2 for quantities less than 30,000 ft^2, and $8.33/ft^2 for quantities over 30,000 ft^2. Hauling, equipment, placement costs, and subgrade preparation will add approximately $0.20 to $0.40/ft^2 to this cost. Recovery costs are approximately $0.40/ft^2.

The steel mats rutted under traffic and separated at two locations. Some of the rutted panels are unusable. The performance of the aluminum mats was excellent.

Membrane-Encapsulated Soil Layer (MESL)

This project tested the use of plastic and fabric membranes to encapsulate a layer of subgrade soil. The plan was to encapsulate a 6- to 8-in lift of subgrade soil with various types of membrane fabric. Three different types of membrane material were used: a 6-mm black polyethylene film, a 20-mm nonwoven geotextile (Fibretex), and a 36-mm reinforced chlorinated polyethylene (RCP). A CRS-2 emulsion was used to tack the various membranes together and bind the chip seals. A 3/8-in crushed rock was used for the chip seal and a 1/2-in crushed rock was used for a surface rock layer.

The 6-mm polyethylene material was unsuitable for the project. It had no tensile strength, and tore and punctured easily. In addition, the asphalt emulsion and chips did not adhere to it when it was used as the top layer of the encapsulation. In fact, as a result of recreational and timber haul traffic during the first month of operation, the chips tore the membrane severely and the wearing surface was therefore lost. The other two materials, the geotextile and the RCP, performed acceptably.

The membrane materials cost $0.56/ft^2 for the 36-mm reinforced polyethylene, $0.051/ft^2 for the 20-mm Fibretex geotextile, and $0.029/ft^2 for the 6-mm polyethylene. The construction cost, including equipment, labor, and materials was as follows:

36 mil material: $11.52/yd^2
6 mil material: $ 4.60/yd^2
20 mil material: $ 4.50/yd^2

Lignin Sulfonate Soil Stabilization

The use of lignin sulfonate, a by-product of the pulp and paper industry, as a stabilizer for subgrade soil was tested on this demonstration project. The project was constructed in 1984 using the lignin to stabilize a subgrade soil that was subsequently covered with a chip seal for a wearing surface.

At least three types of lignin sulfonate are available on the market and in use as soil stabilizing agents or dust abatements: ammonium lignin sulfonate, calcium lignin sulfonate, and sodium lignin sulfonate. Calcium lignin sulfonate was used as a stabilizer in this project. It was the least expensive type and no conclusive evidence was found to indicate that it would perform much differently than the other types. The subgrade for the project varied from a clayey sand (SC) to a sandy gravel (GW) soil derived from decomposed granite.

The project has performed very well to date and no problems have been experienced. It has provided good traction and bearing capacity and has had a relatively smooth, high-quality riding surface. The costs of construction were $1.20/yd^2 for lignin stabilization and $1.10/yd^2 for the chip seal, for a total of $2.30/yd^2.

EVALUATION OF PROJECT RESULTS

The suitability of a particular surfacing system depends on environmental and economic factors, subgrade, unique requirements for construction and maintenance, limitations on its use, and projected loading conditions.

About half of the alternate surfacings evaluated required special technology, equipment, or expertise to construct and maintain. These requirements are summarized in Table 4.

Some of the alternate surfacings that were properly constructed and maintained performed adequately. Specific aspects

TABLE 4 UNIQUE REQUIREMENTS OF THE ALTERNATE SURFACING SYSTEMS

Potential Surfacing Types	Construction, Recovery, and Maintenance Technology	Special Equipment
Wood and bark chip	The same as aggregate roads	Chipper
Chemical stabilization	Special construction methods necessary special expertise in mixing and spreading	Pulva-mixer or twin disk harrow, distributor tanker
Geotextile or geogrid separation	Special construction methods necessary	None required
Marginal aggregate	None required	None required
Sand-sealed subgrade	None required	None required
Metal mats	None, mats easily pieced together in field	Forklift or truck-mounted crane, pressure washer, mobile welder
Reusable aggregate without geotextile separation	None required	None required
Reusable aggregate with geotextile separation	Requires special technology for the recovery of materials	Sewing machine, recovery system
Geoweb stabilization (expandable grids)	Technology requires special knowledge for geoweb placement, filling, leveling, and compaction	None required
Membrane encapsulated soil layer	Technology is unique in laying fabric, applying emulsion, compacting, and sealing joints	Asphalt sprayer

of these surfacing materials are discussed in Table 5. The potential applications of these materials are described in Table 6.

CONCLUSIONS AND RECOMMENDATIONS

The results of the study indicate that alternate surfacings can be used in many situations. Based on the total cost of construction and maintenance and the limited performance data, the most promising surfaces, in no particular order, for typical logging roads are as follows:

- Wood and bark chips,
- Chemical stabilization,
- Marginal aggregates,
- Reusable aggregates with or without inexpensive geotextiles,
- Sand-seal membrane, and
- Steel mats for emergency situations and short projects.

The characteristics of the log haul, the subgrade, and the availability of materials dictate which materials are most cost-effective.

Conclusions

The following specific conclusions can be drawn from this study:

- Biodegradable materials, such as bark and wood chips, can perform suitably on temporary roads, especially logging roads, because they are inexpensive and available. These materials are recommended for short roads (less than 1 mi) with low speeds and moderate grades.
- Admixture stabilization materials, such as lime, Portland cement, emulsified asphalt, fly ash, sodium chloride, calcium chloride, magnesium chloride, and lignin sulfonate, can be used to upgrade local soils and marginal aggregates. The selection of specific additives to stabilize the materials depends on the

TABLE 5 PERFORMANCE AND ECONOMIC EFFECTIVENESS OF ALTERNATE SURFACINGS

Material	Performance	Economic Effectiveness
Biodegradable materials	Limited rutting, low dusting levels	Economic, easily constructed
Chemical stabilization	Some bearing failures, limited rutting and potholing	Very economic
Geotextile, geogrid separation	Some rutting, strength increase under fabric	Geotextile economic; geogrid too costly
Marginal aggregate	Differs by subgrade soil and material; early rutting possible	Low construction costs, but high maintenance costs possible
Geoweb Stabilization	Half of surface cracked, potholed, rutted after 3 years; surface bleeding prevalent	Cost effective in certain locations
Reusable/Recycled aggregate with geotextile	Performs as well as quality aggregate	Economically effective if light textile used, but not recovered
Metal mat surface	Slippery when wet; panels and connectors bend with use on poor subgrade	Materials costs very high; economically effective for bridging wet or soft spots
Membrane encapsulated Soil layers	Controls frost heave; maintains strength; controls moisture content	Cost effective for poor, moisture sensitive soils

TABLE 6 POTENTIAL APPLICATIONS OF THE ALTERNATE SURFACING SYSTEMS

Potential Surfacing Type	Potential for Future Use[a]	Applicable Situation
Wood and bark chips	High	Subgrades with wood and bark chips available
Chemical stabilization	High	Depends on soil type; clayey soils best
Geotextile or geogrid separation	High	Weak, wet, and fine-grained subgrades
Sand- or chip-sealed subgrade	Low	May not work on weak subgrades (CBR <15)
Metal Mats	Low (Alum.) Medium (Steel)	Economical only on short sections
Reusable aggregate without geotextile separation	Medium	Firmer subgrade to control rutting and intrusion of fine material
Reusable aggregate with geotextile separation	Medium	Soft low strength subgrades, may increase in strength
Membrane encapsulated Soil Layer	Low	Economical only on short critical sections
Geoweb stabilization	Low	Uniform sands and critical sections
Lignin sulfonate soil stabilization	High	Dry climates, requires a surface seal

[a]High: up to 80 percent of USFS local mileage; medium: up to 50 percent of USFS local mileage; low: less than 10 percent of USFS local mileage.

subgrade material types and the availability and costs of the additives in the area. Many such projects have been constructed on temporary and intermittent-use roads. This is likely to be one of the most economical solutions.

- Marginal aggregates are widely available and have a low initial cost. They are recommended for use on roads with short project lives (1 to 3 yrs).
- Conventional geotextiles and extruded plastic grids can be used to stabilize weak soils. They are recommended for situations in which rock is costly because they reduce the amount of rock required.
- Sand-sealed subgrades are recommended for sections that have a good, firm subgrade (CBR greater than 15; R value greater than 40). The soft nature of the sand seal probably precludes its use on steep grades or sections with sharp curves. Therefore, the use of this material should be limited to roads with firm subgrades, moderate grades, and a short project life (1 to 3 yrs).
- Steel and aluminum mats are recommended for use on short sections of road or for emergency situations because of their high initial cost.
- The recovery and reuse of good-quality aggregates separated with a layer of geotextile may not be economical because of the high cost of the geotextile. The recovery and reuse of good-quality aggregates without a layer of geotextile may reduce construction costs. It is expected that about 70 to 75 percent of the aggregates could be recovered each time.
- The membrane-encapsulated soil layer (MESL) concept should only be considered for use in regions with very fine-grained soils, a high moisture content, and a lack of quality aggregate.
- Geowebs are not economical for temporary or intermittent-use roads when compared to crushed aggregates because of the high initial cost of materials.

Recommendations

Much work remains to be done before the economic feasibility of alternate surfacings can be completely understood. This study was based on a summary of existing literature and the construction and evaluation of 11 demonstration projects in which various alternate surfacings were tested. No conclusions should be drawn from the results of this study that would lead to the application of these materials across the entire country without first determining their suitability. Additional study of these materials in low-volume, temporary, or intermittent-use road applications would be of tremendous value.

ACKNOWLEDGMENTS

The need for this study was identified at the national level of the U.S. Forest Service. The funds for the study were provided by the Washington Office of the U.S. Forest Service. The work was performed under a contract with the Equipment Development Center of the U.S. Forest Service in San Dimas, California. Their support is gratefully acknowledged.

Special thanks are extended to Ed Gililland (COR) of the Equipment Development Center for his assistance in all aspects of this study.

The views expressed in this paper are those of the authors, and do not necessarily reflect the official views of the sponsors. The authors are solely responsible for the accuracy of the material presented.

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Evaluation of Alternate Surfacings for Forest Roads

M. B. TAKALLOU, R. D. LAYTON, AND R. G. HICKS

A procedure is presented to evaluate potential alternate surfacings for forest roads and to compare their cost-effectiveness with crushed aggregate road surfacings. The alternate surfacing types considered include (a) biodegradable materials (wood and bark chips), (b) chemical stabilization, (c) geotextiles, (d) marginal aggregates, (e) sand-sealed subgrade, (f) metal mats, (g) reusable aggregates, (h) membrane-encapsulated soil layer, and (i) geoweb stabilization. The use of the alternate surfacings presents an attractive alternative when compared to the use of crushed aggregates from the standpoint of a savings in aggregate, utilization of the waste materials, and most important, cost-effectiveness. A two-step evaluation procedure was developed and recommended to evaluate the alternate surfacing types. The preliminary evaluation step is to screen various alternative materials based on their characteristics, limitations, and availability. Those materials that have the strongest potential to be effective are identified. The next step is to evaluate the cost-effectiveness of these alternate surfacings to determine which materials have the least total present worth of life-cycle costs. This methodology is applied to two examples of low-volume logging roads. The sensitivity of the decision of which material is most suitable is demonstrated by important variables and costs. Finally, a user-friendly computer program that demonstrates the evaluation methodology is proposed to aid in the decision-making process of whether to use aggregate or alternate surface to build a road.

The USDA Forest Service, Bureau of Land Management (BLM), and other agencies or industries traditionally place crushed rock, pit run, or select borrow material on intermittent-use or temporary-service roads when a surface is warranted to haul timber or for other resource activities. When the timber haul, mining, or other activities are completed, the surfacing and the capital investment that these activities represent lie idle for periods of up to 20 years. A description is provided of the evaluation of selected alternate surfacing systems that can reduce the total investment in intermittent-use and temporary-service roads. For more information on this topic, refer to "An Overview of Alternate Surfacings for Forest Roads," by M. B. Takallou et al., which can be found elsewhere in this publication. The alternatives considered include surfacing systems that are capable of being moved as the hauling, mining, or other activities move; degrade soon after use; significantly reduce the amount of surfacing required; or make better use of available resources to reduce construction costs.

The overall purpose of this study is to provide the background to evaluate alternate surfacing systems and compare their cost-effectiveness with that of quality aggregate-surfaced roads. The methodology used to analyze and evaluate the effectiveness and economic viability of potential surfacing systems is described.

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This methodology is then applied to two examples of low-volume logging roads. The sensitivity of the decision of which material is most suitable is demonstrated by important variables and costs.

Finally, a user-friendly computer program (ALTSURF) that demonstrates the evaluation methodology is proposed to aid in the decision-making process of whether to build a road with alternate or aggregate surfaces.

BACKGROUND

Potential Surfacing Types

Surfacing systems that can potentially serve as alternates to typical quality aggregate surfacing types for logging and forest service roads include (a) biodegradable materials (wood or bark chips), (b) chemical stabilization, (c) geotextile or geogrid separation, (d) marginal aggregate, (e) sand-sealed subgrade or chip seal on native subgrade, (f) metal mats, (g) reusable aggregate without geotextile separation, (h) reusable aggregate with geotextile separation, (i) membrane-encapsulated soil layer (MESL), and (j) geoweb stabilization (expandable grids). Each of these surfacing types is described in Table 1.

Criteria for Alternate Surfacing Applications

Alternate surfaces are generally considered suitable for use on local or spur roads that are used on a temporary or intermittent basis. Alternate surfacings can be employed on roads that primarily serve logging traffic of 50 to 100 vehicles per day (vpd), and require a life of 1 to 5 years for the surfacing. Projects that are relatively close to other projects can facilitate reuse of the surface materials. Roads on subgrades of a low strength can use alternate materials if they provide an increase in strength.

TABLE 1 GENERAL DESCRIPTION OF POTENTIAL SURFACING TYPES

Potential Surfacing Type	General Description
Biodegradable materials	Bark chips, wood chips, and sawdust have been used to stabilize roads built in slide-prone areas. Bark and wood chips are an inexpensive method of constructing temporary logging roads (*1*).
Chemical stabilization	Lime, Portland cement, emulsified asphalts, fly ash, sodium, calcium or magnesium chloride, and lignin sulfonate can be used to alter the following soil properties: strength, compressibility, permeability, volume stability, plasticity, and durability (*1, 2*).
Geotextile and geogrid	Geotextiles are mainly used in roads to separate the surface or base from poor subgrade. Geogrids, which are high-strength polymer structures, stabilize weak soils (*3*).
Marginal aggregates	Marginal aggregates are those that do not meet standard specifications for road use and may require some type of additive or special treatment. The types of marginal aggregates include cinder, pumice, rhyolite, coquina, decomposed sandstone, marginal sand, pit-run gravel, sand-clay shale, baked shale, chert, and marine basalts (*1*). Marginal aggregates can be used for logging roads that require a life of approximately 1 to 3 years.
Sand-sealed subgrades	One application of emulsified asphalt on natural subgrade followed with a sand topping, which waterproofs a subgrade soil. Sand-sealed subgrades are most effective in areas that have good subgrade and high-cost rock, but that have a plentiful source of low-cost sand (*1, 4*).
Metal mats	Metal mats are surface panels prefabricated from materials such as aluminum and steel. The mats come in various panel sizes and can be connected together to form a continuous road surfacing (*5*).
Reusable aggregate without geotextile separation	The recovery and reuse of high-quality aggregate on several projects may lower the construction cost. The reuse of the high-quality aggregate can be most effective for the projects that have high rock cost, short duration of logging, short hauling distances between projects, and projects about equal in size. About 70 to 75 percent of the aggregate can be recovered each time for future use (*1, 4*).
Reusable aggregate with geotextile separation	Similar to the former description, except that a light geotextile is used to separate subgrade and surface materials to prevent the loss of aggregate into the subgrade. About 90 to 95 percent of the aggregate can be recovered each time for future use. (*1, 4*).
Membrane-encapsulated soil layer (MESL)	The MESL concept maintains the moisture content of the subgrade soils at a desired level by encapsulating the soil in a waterproof membrane to prevent water infiltration. The MESL should generally be used with fine-grained soils that are susceptible to strength loss if wetted (*6*).
Geoweb stabilization (expandable grids)	Geoweb opens like an egg crate divider into an 8-ft by 20-ft panel. The expanded panels are set in place, and the cells are filled with sand. The sand is compacted and the surface is sprayed with a liquid asphalt (*7*).

Regions in which aggregate is costly or not readily available because of rugged mountains or long haul distances can benefit from the use of alternate surfacing systems.

EVALUATION METHODOLOGY FOR ALTERNATE SURFACINGS

A methodology is described in this section to evaluate the effectiveness and economic viability of potential surfacing systems and compare them with conventional crushed-aggregate roads. The evaluation methodology is structured as a two-step process. A preliminary evaluation step is taken to screen the various alternative materials based on their characteristics, limitations, and availability to determine if they can be used. An economic evaluation step is then taken to perform an in-depth economic evaluation of the total costs of each potential alternative identified in the preliminary evaluation step.

Preliminary Evaluation Step

The potential use of a particular surfacing system depends on a variety of environmental, subgrade, and loading conditions that could limit or preclude its use. This step in the evaluation process is taken to screen the various materials to determine if they meet the objectives and needs of the road projects.

This step in the evaluation process is depicted in Figure 1 and is described as follows:

- *Define Project Objectives.* The objectives to be achieved by the surfacing must be defined at the start. Some example objectives are (a) a surfaced road to serve logging only, (b) an all-weather surface, (c) temporary or intermittent use of the road for a 1- to 5-yr period, (d) economic alternative to a permanent road, and (e) low initial cost construction.
- *Determine Available Materials.* Any materials that are available for use on the projects, such as slash for a wood-chip surfacing, must be identified.
- *Gather Data.* Data must be collected and assembled concerning the project and possible materials, including the volumes and loading conditions, subgrade type and strength, season of use and expected climatic conditions, materials costs, and other pertinent information.
- *Screen Possible Alternate Surfacings.* As shown in Figure 1, those materials that have a strong potential of being used can be identified through a comparison of the materials' properties, characteristics, limitations, unique requirements, and potential applications with the project's objectives, needs, and characteristics. This screening process is used to identify which alternate surfacings have the strongest potential of being effective but does not account for their relative economic efficiency. The following are major factors that must be considered to identify potential surfacing alternates in the preliminary evaluation step.
  - *Material Properties and Characteristics.* Many material properties are needed to evaluate the adequacy, economy, and suitability of alternate surfacings. Some of these include the materials' size, weight, surface thickness, cost, performance, and availability. The material properties and characteristics may give adequate information in many situations to determine whether an alternate surfacing material can be used or not.
  - *Limitations and Constraints.* Certain constraints on some alternate surfacings considered may limit their use. For example, some of the materials may not be applicable because of the nature of the subgrade, season and climate, geometrics of the roadway, useful life, project length, and cost. Some limitations that have the greatest influence on the selection of the alternate surfaces are summarized in Table 2.
  - *Special Requirements for Equipment and Labor.* The equipment and labor required to construct the surface must be available. Specific construction and maintenance requirements for each alternate surfacing are listed in Table 3. Special equipment, technology, or expertise may be difficult to provide continuously on a long-term basis if not used frequently.
  - *Availability.* The materials must be available in the area of use for them to be economical. The final measure of a material's availability is the cost of the material, including its purchase price and the cost to ship it to the job site.
  - *Budget Constraints.* Although the source and amount of financing for a road surface material do not affect

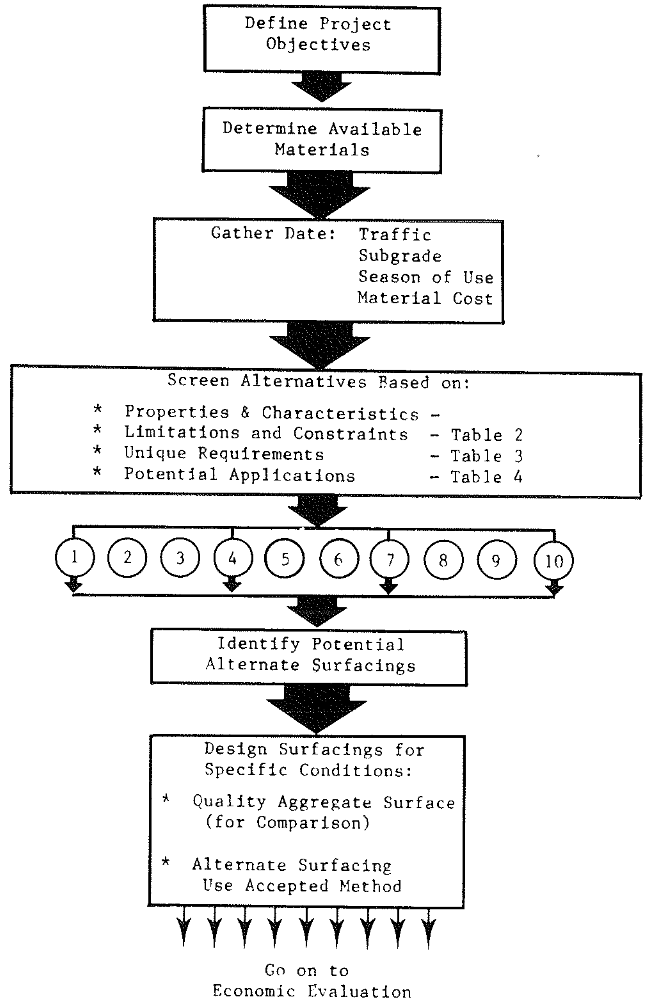

FIGURE 1 Preliminary evaluation step.

TABLE 2 LIMITATIONS OF THE ALTERNATE SURFACING SYSTEMS

Potential Surfacing Types	Subgrade Soil Type	Geometrics of the Roadway	Expected Life
Wood and Bark Chip	None	Not recommended on steep grades	1 to 3 years
Chemical Stabilization	Depends on chemical	None	3 to 5 years
Geotextile and Geogrid Separation	Effective on weak subgrade	None	Sames as quality aggregate
Marginal Aggregate	None	None	2 to 3 years
Sand Seal Subgrade	Not on weak subgrades	Not recommended on steep grades or sharp curves	3 years
Metal Mats	Not recommended on very weak soils (CBR 3)	Not recommended on steep grades or sharp curves	5000 passes or 25 MMBF
Reusable Aggregate without Geotextile	Recommended for firmer subgrade	None	Same as quality aggregate
Reusable Aggregate with Geotextile Separation	None	None	Same as quality aggregate
Membrane Encapsulated Soil Layer (MESL)	Works best on organic clay, wet and fine-grained soils	Not recommended on steep grades	Unknown
Geoweb Stabilization (Expandable Grids)	Works best on weaker sand soils	Not recommended on steep grades	Unknown
Lignin Sulfonate	Clayey sand (SC) to sandy gravel	None	3 to 5 years

the comparative economic analysis of alternatives, they could become considerations in the decision-making process if a budget constraint existed. The preferred alternative may appear to be attractive on a life-cycle basis, but the initial purchase and construction cost may be too high for the budget. A detailed economic evaluation is made in the next step.

— *Promising Applications.* A summary is provided in Table 4 of the potential applications of the alternate surfacing system, the degree of quality control needed to ensure success, and applicable situations for which the surfacing type should be employed. The degree of quality control depends on the allowable variations in compaction thickness and placement practices, for example.

• *The Design of Surfacings for Specific Conditions.* Each of the potential alternative surfacings must be designed to meet the loading requirements, subgrade conditions, and environmental conditions using accepted design approaches.

Economic Evaluation Step

Those alternatives that meet the physical and financial requirements are evaluated in the economic evaluation step of the process. The economic evaluation of alternate surfacings is based on the least total life-cycle cost. The analysis period is defined as the life of the set of planned projects instead of the life of the surfacing. The total costs include the costs of construction, maintenance and repair, recovery and replacement, and storage. These costs are converted to a comparable economic basis by reducing all costs to their present worth. The elements of this step are depicted in Figure 2.

• *Estimate Project's Costs.* The construction, maintenance, recovery, storage, and the salvage values for each of the projects in the set are estimated. The set of projects includes all projects in which material use or reuse is planned.

• *Convert Costs to Present Worth of Costs.* The expected costs and salvage value for all projects in the set of projects are converted to their present worth by the appropriate engineering economic factor for the period, n, and the accepted discount rate, i.

• *Compute Present Worth of Costs for Analysis Period.* The sum of the present worth of all costs incurred on the project over their analysis period is computed.

• *Compare Present Worth of Various Alternatives.* The present worth of the various surfacing alternatives identified by the preliminary evaluation are compared with the costs for a normal, aggregate-surfaced road, and with each other.

• *Select Surfacing With Least Present Worth.* The surfacing alternative, or normal aggregate surfacing, with the least present worth is selected as the most economical.

TABLE 3 UNIQUE REQUIREMENTS OF THE ALTERNATE SURFACING SYSTEMS

Potential Surfacing Types	Construction, Recovery, and Maintenance Technology	Special Equipment	Special Expertise
Wood and Bark Chips	The same as aggregate roads	Chipper	None required
Chemical Stabilization	Requires special mixing equipment	Pulva-mixer or twin disk harrow, distributor tanker	Special expertise needed to spread and mix additives
Geotextile or Geogrid Separation	Special construction methods necessary	None required	None required
Marginal Aggregate	None required	None required	None required
Sand- or Chip-Sealed Subgrade	None required	None required	None required
Metal mats	None, mats easily placed together in field	Fork lift or truck-mounted crane, pressure washer, mobile welder	None required
Reusable Aggregate without Geotextile Separation	None required	None required	None required
Reusable Aggregate with Geotextile Separation	Requires special technology for the recovery of the materials	Sewing machine and special recovery system	Trained laborers need to sew the fabric around recovery beam
Geoweb Stabilization (Expandable Grids)	Special knowledge needed for subgrade preparation, geoweb placement, filling, and compacting the surface	None required	Trained laborers needed for parts of construction
Membrane Encapsulated Soil Layer (MESL)	Special knowledge needed for laying the fabric, applying emulsion, and compaction	None required	None required

TABLE 4 POTENTIAL APPLICATIONS OF THE ALTERNATE SURFACING SYSTEMS

Potential Surfacing Types	Potential for Future Use*	Degree of Quality Control**	Applicable Situation
1. Wood and Bark Chips	High	Low	Any subgrade with wood and bark chips available
2. Chemical Stabilization	High	High	Depends on the chemicals, clayey soils best
3. Geotextile or Geogrid Separation	High	Medium	Wet and fine-grained subgrades Weak subgrades
4. Marginal Aggregate	High	Low	Any subgrade
5. Sand- or Chip-Sealed Subgrade	Low	Medium	May not work on weak subgrade (CBR < 15)
6. Metal Mats	Low (Alum.) Med. (Steel)	High	Economical only on short sections
7. Reusable Aggregate without Geotextile Separation	Medium	Medium	Firmer subgrade to control rutting and intrusion of fines into the aggregate
8. Reusable Aggregate with Geotextile Separation	Medium	Medium	Soft subgrade of low strength may experience strength increase
9. Membrane Encapsulated Soil Layer (MESL)	Low	High	Economical only on short critical sections
10. Geoweb Stabilization	Low	High	Uniform sands and critical sections
11. Lignin Sulfonate Soil Stabilization	High	Low	Dry climates, requires a surface seal

* High -applicable for up to 80% of USFS local mileage
Medium -applicable for up to 50% of USFS local mileage
Low -applicable for less than 10%

** High -good technical supervision
Medium -moderate supervision
Low -little supervision

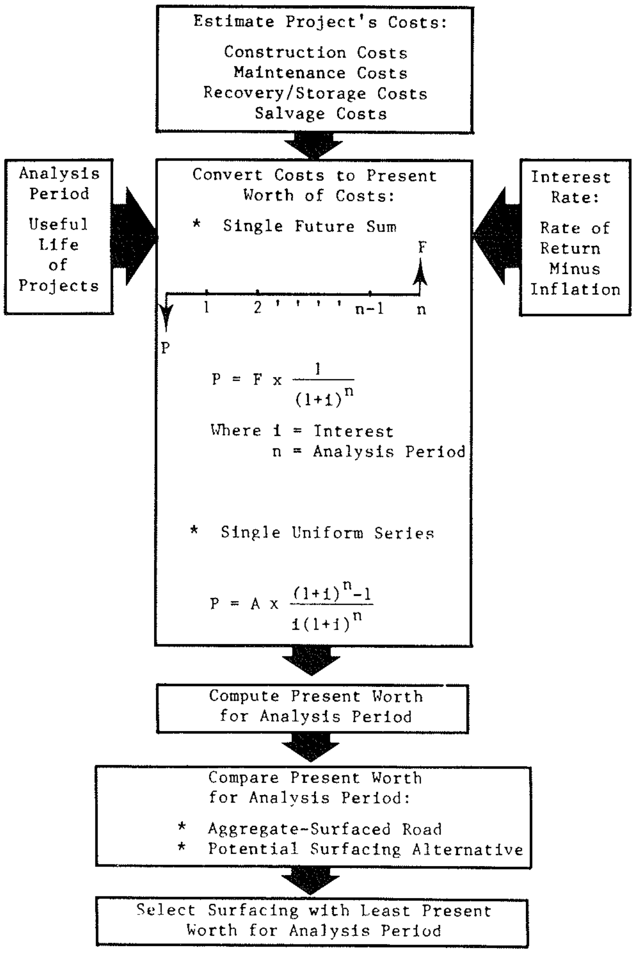

FIGURE 2 **Economic evaluation step.**

Estimate Costs for Each Alternate Surfacing

For each alternative, the following costs must be estimated:

- *Initial Construction Costs.* The analyst should be aware that the term "construction cost" refers only to the cost of material, equipment, labor, and overhead used for placement of the surface, including any extra-fine grading required by the material. The normal subgrade preparation costs, which include right-of-way, excavating, clearing and grubbing, grading, and compaction costs, are not included in the construction cost. Typical construction costs for alternate surfacings are given in Table 5.
- *Maintenance Costs.* Maintenance costs include major and routine maintenance costs.
  - Major maintenance cost includes the costs of regraveling or reconstruction that brings the surface back to its original constructed condition.
  - Routine maintenance costs include the cost to correct surface distress as it occurs, instead of at specified periods of time after construction. This includes filling the potholes, replacing the broken pieces of metal mats, patching, and so forth.
- *Recovery Costs.* At the end of a period of use, some materials can be recovered for later use or use on another road.
- *Vehicle Operating Costs.* The differences in the vehicle operating costs, which include fuel, engine oil, tires, and depreciation, are very slight for alternate surfaces. Therefore, they are not considered in the economic evaluation of alternate surfacings.

The ALTSURF computer program, which is described in more detail later, can be used to estimate the total construction, maintenance, and recovery costs of the alternate surfacings.

Important Values for Economic Analysis

In order to convert costs to their present worth, several key items of information need to be determined, including the interest rate, analysis period, and salvage values.

- *Interest Rate.* The interest rate should reflect the real cost of capital, which is the current acceptable rate of return minus the inflation rate.
- *Analysis Period.* The analysis period is taken as the life of a set of projects for which the loading conditions and scheduling are known. This then determines the useful life of the set of projects.
- *Salvage Value.* The salvage value is the economic residual value of the surface material at the end of the analysis period for the project (*9*). The salvage value of the nonrecoverable materials is assumed to be zero. The salvage value of the recoverable materials for sale or use on another project can be estimated based on its anticipated remaining life. The following equation can be used to estimate the salvage value (*9*).

$$SV = \left(1 - \frac{L_A}{L_E}\right) C \qquad (1)$$

where

SV = salvage value of the surfacing alternative in dollars;
L_A = cumulative life of all projects in the project set, in years or repetitions of 18-kip single-axle loads;
L_E = expected life of the surfacing alternative, in years or repetitions of 18-kip single-axle loads; and
C = initial cost of the materials for the surfacing alternative in dollars.

Compute Present Worth of Costs for Each Alternative

The present worth of costs for each surfacing alternative should be calculated using the interest rate selected and an analysis life equal to the service life of the set of projects. Salvage values should be calculated based on Equation 1.

Equations 2 and 3 are the basic equations for computing the total present worth of the life-cycle costs. These equations do not include the vehicle operating costs.

Nonrecoverable

$$PW = C + M_1 \left(\frac{1}{1+i}\right)^{n_1} + \text{----} + M_k \left(\frac{1}{1+i}\right)^{n_k} \qquad (2)$$

TABLE 5 TYPICAL CONSTRUCTION COSTS FOR ALTERNATE SURFACINGS

Surfacing Material	$/Mile/Lane[a] Average	$/Mile/Lane[a] Range	$/ft² Average	$/ft² Range
Crushed Aggregate[c] (Comparison base)	38,000	13,000 - 70,000[b]	0.60	0.20-1.00
Wood and Bark Chip[d]	22,750	10,500 - 35,000[b]	0.36	0.17-.55
Chemical Stabilization	25,500	19,000 - 32,000	0.40	0.30-0.50
Geotextile and Geogrid Separation	12,750 plus aggregate surface cost	6,500 - 14,000 plus aggregate surface cost	0.20	0.10-0.30
Marginal Aggregate[c]	31,750	6,500 - 57,000[b]	0.50	0.10-0.90
Sand Seal	15,000	10,000 - 20,000[b]	0.24	0.16-0.32
Geoweb Stabilized	158,000	126,000 - 190,000	2.5	2.0-3.0
Reuse/Recycled Aggregates[c]:				
a. Construct	45,000	20,000 - 70,000[b]	.7	0.30-1.10
b. Recover[e]	9,000	6,000 - 12,000[b]	.15	0.10-0.20
Metal Mats:[f]				
a. Steel	110,500	95,000 - 126,000	1.75	1.5-2.0
b. Aluminum	565,000	500,000 - 630,000	9.0	8.0-10.0
Membrane Encapsulated Soil Layer (MESL)	48,000	32,000 - 64,000[b]	0.75	0.50-1.0

Assumptions:
[a]assumes lane width of 12 feet
[b]variation is due to the transportation costs of the materials
[c]assumes 12-inch surface thickness
[d]assumes 18-inch surface thickness
[e]assumes 70% of the aggregates can be recovered
[f]not in production. Estimate is based on the material cost from surplus

Recoverable

$$PW = C + M_1 \left(\frac{1}{1+i}\right)^{n_1} + \cdots + M_k \left(\frac{1}{1+i}\right)^{n_k} + R\left(\frac{1}{1+i}\right)^{n_r} - S_A\left(\frac{1}{1+i}\right)^{n_r} \quad (3)$$

where

PW = present worth or present value of total life-cycle cost;
C = initial construction cost;
M_k = cost of the k^{th} repair or routine maintenance;
R = recovery and rehabilitation cost;
S_A = salvage value;
i = interest rate;
n_k = number of years from present to the k^{th} maintenance, rehabilitation, recovery activity;
n_r = number of years from the present to the recovery date; and

$\frac{1}{(1+i)^n} = (P/F, i, n)$ = the single payment present factor.

EXAMPLES OF EVALUATION METHODOLOGY

Two example road projects are evaluated to demonstrate the evaluation methodology. All relevant life-cycle costs are included in the determination of the total costs. The potential surfacings are all designed to provide an equivalent level of service for the timber transport accommodated.

Example One

A 1-mi-long, 14-ft-wide local road is to be constructed for the removal of 5.0 million board feet of timber. The road characteristics, requirements, and design data are summarized in Table 6.

Preliminary Evaluation

Employing the screening process described earlier (Figure 1), the alternate surfacings that have a strong potential of being effective are selected as follows:

- *Identify Project Objectives*: Construction of temporary local road for logging access and one season of use.

TABLE 6 DESIGN DATA FOR EXAMPLE ONE

Length of the road (ft)	5,280
Width of road (ft)	14
Timber volume (MMBF)	5.0
Project life (months)	6.0
Maximum grade (%)	12
Side slopes	2:1
Soil type	Silty-sand (SMd)
CBR at 95 percent of AASHTO T-180	8
Design aggregate thickness[a]	8 in
Design wood and bark chip[b]	12 in

[a]Using USFS Chapter 50, simplified design char. for aggregate surfaced road (*10*), pp. 50-71.
[b]One inch of aggregate = 1.5 inches of wood and bark chips.

- *Identify Available Alternate Surfacing*: All alternate surfacings are available.
- *Gather Data*: Project data are summarized in Table 6.
- *Screen Alternatives*: The surfacing alternatives are screened based on their properties, limitations, and unique requirements relative to the project characteristics and requirements. This screening process is documented in Table 7. As a result of this initial screening, wood chips, chemical stabilization, and marginal aggregates are the alternatives that have a strong potential to be effective.

Economic Evaluation

The three potential alternate surfaces, and a crushed aggregate surface, are next evaluated economically. First, detailed cost estimates are made for the construction and maintenance of each material. This includes the costs of the material, placement, and haul.

The total present worth of life-cycle costs for all surfacings are then calculated. The results of these calculations are shown in Figure 3. The first material evaluated is a conventional crushed-aggregate surface, which gives the base of comparison to usual construction practices. As seen from these results, all alternate surfaces would result in total life-cycle costs lower than a crushed-aggregate surface for this project. The most economical alternative is a chemically stabilized surface.

Example Two

This project has the following characteristics and requirements. Three local road projects are to be constructed for the removal of timber. The roads are 14 ft wide but differ in length, subgrade strength, timber volume to be removed, and the date of construction. The materials from Project 1 can be used for Project 2 and then for Project 3. Quality crushed aggregate must be hauled 30 mi; marginal aggregate and wood and bark chips must be hauled 10 mi. The road characteristics, requirements, and design data are summarized in Table 8.

Preliminary Evaluation

The alternate surfacings that have the strongest potential of being effective are selected by employing the screening process documented in Table 9. As a result of this initial screening, wood and bark chips, chemical stabilization, marginal ag-

TABLE 7 PRELIMINARY EVALUATION FOR EXAMPLE ONE

Potential Surfacing System	Availability	Project Length	Material Cost	Subgrade Type and Strength	Selected Alternative for Economic Evaluation
Wood and bark chip	X	X	X	X	X
Chemical stabilization	X	X	X	X	X
Geotextile and geogrid separation	X	X	X	Subgrade strength[a] does not require geotextile	X
Marginal aggregate	X	X	X	X	X
Sand seal	X	X	X	Subgrade too weak[d]	
Metal mats	X	Too long[b]			
Reusable aggregate without geotextile separation	X	X	X	Subgrade too weak[d]	
Reusable aggregate[c] with geotextile separation	X	X			
Membrane-encapsulated soil layer (MESL)	X	X	Too expensive[e]		
Geoweb stabilization (expandable grids)	X	X	Too expensive[f]		

Note: X = accepted alternatives.
[a]Deleted based on Tables 2 and 4.
[b]Deleted based on Table 2.
[c]Deleted because the alternative project for using the recoverable materials is not defined.
[d]Deleted based on Table 4.
[e]Deleted based on Tables 2 and 4.
[f]Deleted based on Tables 2 and 4.

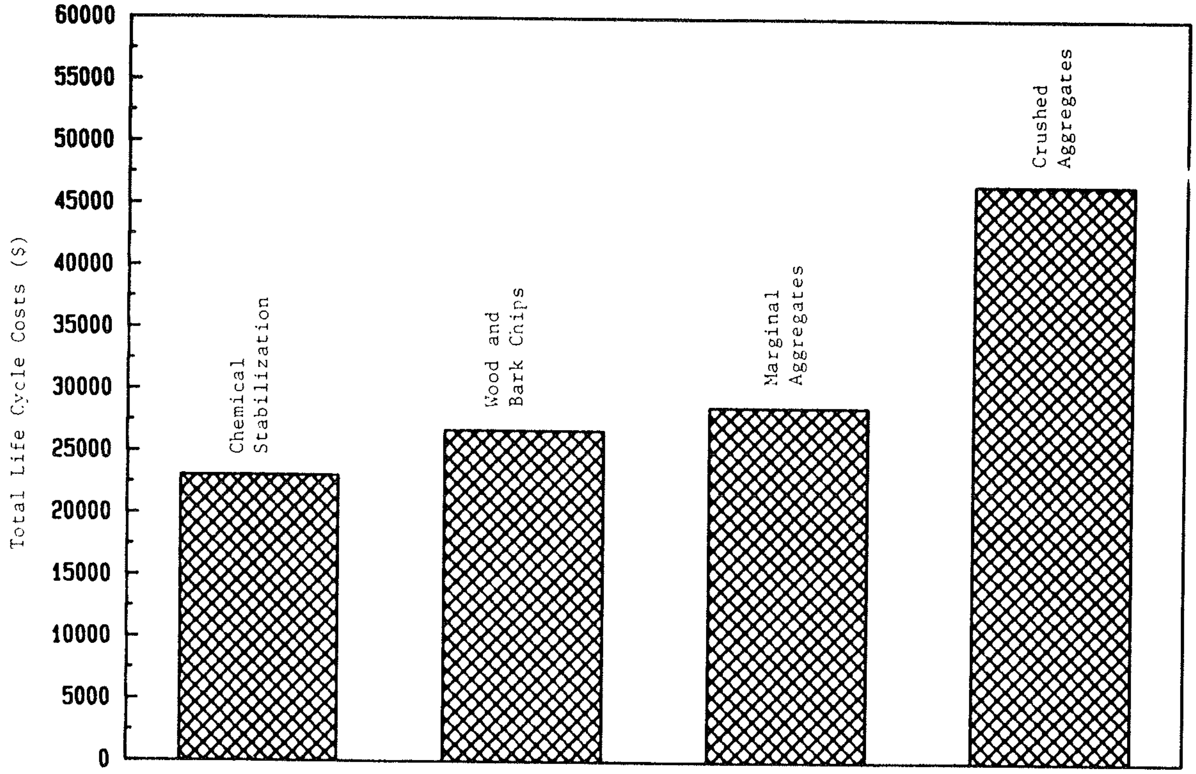

FIGURE 3 Life-cycle cost of alternate surfacings (Example Two).

TABLE 8 DESIGN DATA FOR EXAMPLE TWO

Items	Project 1	Project 2	Project 3
Length of road (ft)	5,280	6,000	4,000
Width of road (ft)	14	14	14
Timber volume (MMBF)	5	8	10
Subgrade type	Silty sand (SMd)	Silty sand (SMd)	Silty sand (SMd)
CBR at 95 percent of AASHTO T-180	8	7	6
Wood and bark chip layer thickness equivalent factor compared to rock	1.5	1.5	1.5
Depth of stabilized layer (in)	6	6	6
Percent of stabilized agent (%)	5	5	5
Compacted unit weight of soil (pcf)	110	110	110
Project life (month)	6	6	6
Aggregate design thickness (in)[a]	7.2 say 8	8.6 say 9	9.8 say 10
Wood and bark chip design thickness (in)	12	13.5	15

[a]Using USFS, Chapter 50, simplified design chart for aggregate surface road (*10*).

gregates, and reusable aggregates with geotextile as a separation layer are the alternatives that have the strongest potential of being effective.

Economic Evaluation

The four potential surfaces, and a crushed-aggregate surface, are then evaluated economically. First, detailed cost estimates are made for the construction, maintenance, and recovery of each type of material. These include the costs of materials, placement, recovery, and haul.

The total present worth of life-cycle costs for all surfacings is then calculated. The results of these calculations are shown in Figure 4. As shown in this figure, all alternate surfaces would result in lower total life-cycle costs than a crushed-aggregate surface. The most economical alternative is a cement-stabilized surface.

TABLE 9 PRELIMINARY EVALUATION FOR EXAMPLE TWO

Potential Surfacing System	Availability	Project Length	Material Cost	Subgrade Type and Strength	Selected Alternative for Economic Evaluation
Wood and bark chip	X	X	X	X	X
Chemical stabilization	X	X	X	X	X
Geotextile and geogrid separation	X	X	X	Subgrade strength[a] does not require geotextile	X
Marginal aggregate	X	X	X	X	X
Sand seal	X	X	X	Subgrade too weak[d]	
Metal mats	X	Too long[b]			
Reusable aggregate without geotextile separation	X	X	X	Subgrade too weak[c]	
Reusable aggregate with geotextile separation	X	X			
Membrane-encapsulated soil layer (MESL)	X	X	Too expensive[d]		
Geoweb stabilization (expandable grids)	X	X	Too expensive[e]		

Note: X = accepted alternatives.
[a]Deleted based on Tables 2 and 4.
[b]Deleted based on Table 2.
[c]Deleted based on Tables 2 and 4.
[d]Deleted based on Table 4.
[e]Deleted based on Tables 2 and 4.

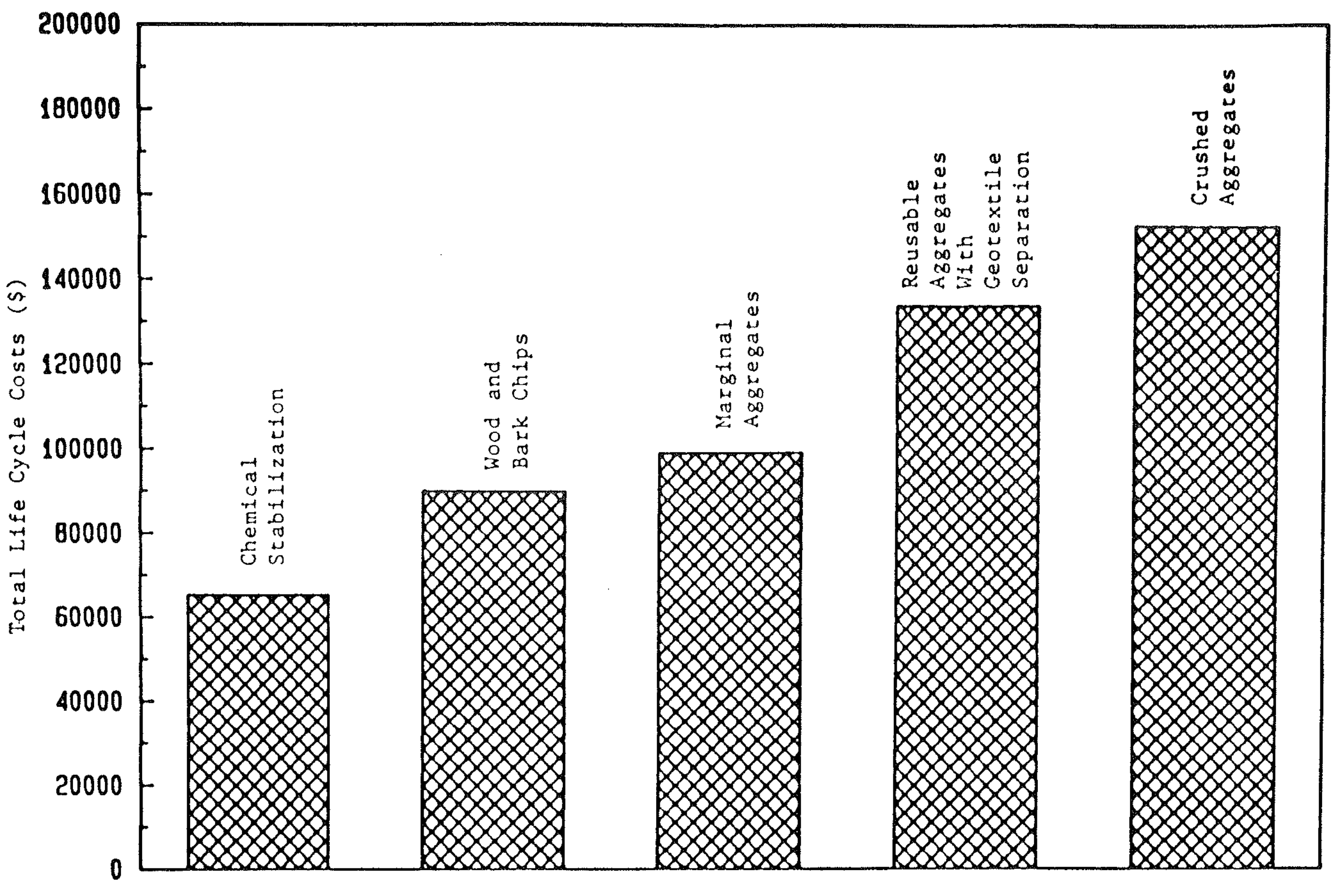

FIGURE 4 Life-cycle costs of alternative surfacings (Example Two).

The results of the comparative evaluation for these examples indicate that alternate surfacings can be more economical than crushed-aggregate surfacings in most of the situations analyzed.

SENSITIVITY ANALYSIS

Economic studies of alternate surfacing materials are based on forecasts of the cost of such items as construction, maintenance, recovery, and salvage values. Most of these costs are estimated based on judgment and construction of a few demonstration projects. Therefore, the results of the analysis show a high degree of uncertainty for various elements of costs or performance.

A sensitivity analysis determines the impact of changes in input variables and provides the decision-maker with information on how the measures of effectiveness, and consequently, the ultimate decision, can vary with these changes. Additional analyses are made using different values of the factors over a range of conditions including the expected value.

Alternate surfacing types for temporary and intermittent-use roads are not expected to be sensitive to either the length of the analysis period or the discount rate because the life of the project or analysis period is very short (1 to 3 yrs) and the expected future maintenance costs are low. However, material costs and haul costs can be very uncertain. These factors are tested for sensitivity.

Crushed-Aggregate Surfacing

The results of the sensitivity analysis of the crushed aggregate in regard to haul distance and material cost based on Example One are shown in Figures 5 and 6. The total life-cycle costs for a crushed-aggregate surfacing under various conditions are calculated. The total life-cycle cost is extremely sensitive to the haul distance of the construction materials. As shown in Figure 5, when the haul distance for aggregate was changed from 10 mi to 60 mi, the total life-cycle cost increased from \$29,220 to \$74,188. The crushed-aggregate surfacing is also highly sensitive to material costs in Example One. When the material cost was changed from \$5/yd^3 to \$20/yd^3 the total life-cycle cost increased from \$47,207 to \$92,175, as shown in Figure 6.

This sensitivity analysis demonstrates that many other alternate surfacings could become more attractive than crushed aggregate if haul distances or material costs are high.

Soil-Stabilized Materials

The results of the sensitivity analysis of the soil-stabilized surface based on Example One in regard to material cost and percent of stabilizing agent are shown in Figures 7 and 8. The stabilized surfaces are sensitive to the cost of the stabilizing agent. As shown in Figure 7, when the material cost was changed from \$50/ton to \$160/ton, the total life-cycle cost increased from \$22,014 to \$36,389. The stabilized surfaces are also sensitive to the percentage of stabilizing agent used. As shown in Figure 8, when the percentage of stabilizing agent was changed from 2 to 10 percent, the total life-cycle cost increased from \$19,074 to \$37,369.

It can again be seen from this sensitivity analysis that other alternate surfacing types may be more attractive than stabilized surfaces. If the amount of stabilizing agent required and the cost of the stabilizing agent, or both, vary significantly, other surfaces may be more economical.

DESCRIPTION OF COMPUTER PROGRAM

A user-friendly computer program, ALTSURF, was developed to perform the estimates of construction, maintenance, and recovery costs, and the economic analysis (*11*). The program analyzes and evaluates the total project life-cycle costs for alternate materials using the least total present worth of costs to determine the best alternative.

The ALTSURF program was written in FORTRAN. It was developed on a Cyber 170, Model 720 mainframe computer and adapted for the Data General and the IBM PC computers. It is structured to run sequentially through the following five interactive menus:

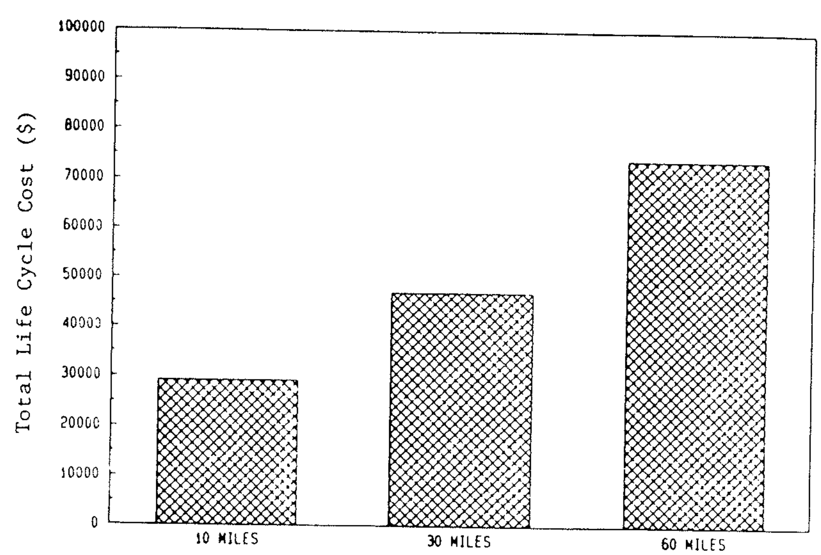

FIGURE 5 Sensitivity of cost of crushed aggregates to haul distance.

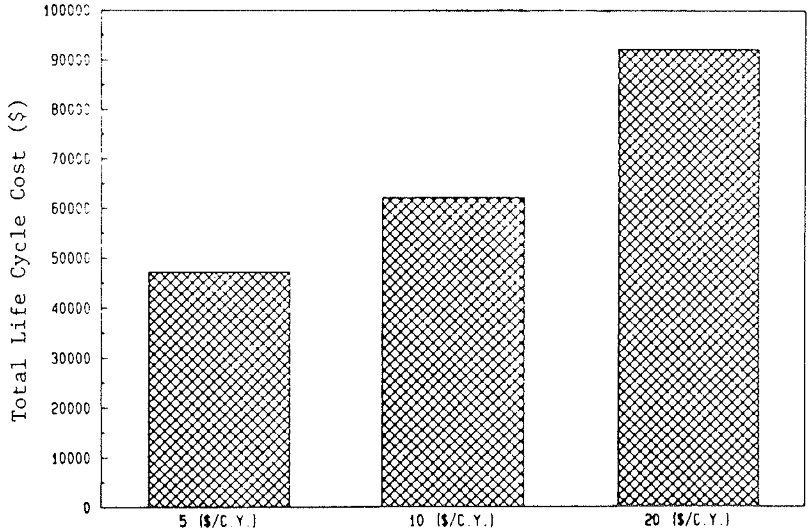

FIGURE 6 Sensitivity of cost of crushed aggregates to material cost.

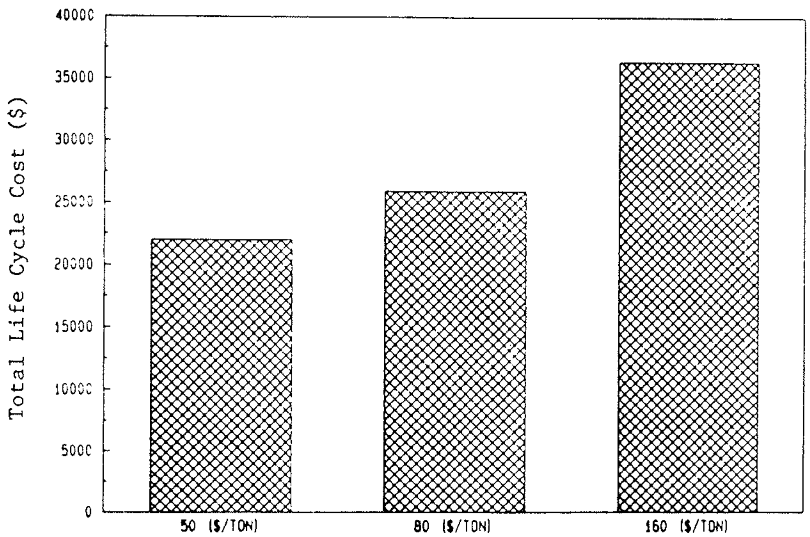

FIGURE 7 Sensitivity of cost of chemical stabilization to the cost of the chemical agent.

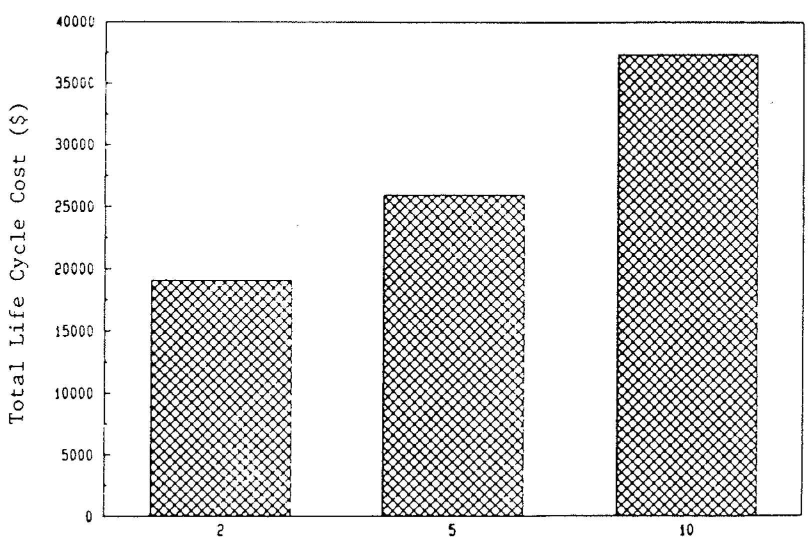

FIGURE 8 Sensitivity of cost of chemical stabilization to the percentage of the chemical agent.

- Menu I: general project information and description.
- Menu II: alternate surfacing system selection and input of evaluation parameters.
- Menu III: for each alternate surfacing system, default values are specified for data on construction and maintenance costs and productivity variables.
- Menu IV: the results of the analysis of total costs for the project are output for each alternate surface selected.
- Menu V: a comparative evaluation of the present worth of each alternate surfacing system.

The menus prompt the user to provide the necessary input to perform the evaluation. These menus provide the inputs to the program, and show the results of the analysis and evaluation. The user only needs to be able to load the program to be able to use it. The program guides the user through all steps.

CONCLUSIONS AND RECOMMENDATIONS

The evaluation of the alternate surfacings must account for the material properties, costs, limitations, and unique requirements of the various materials. Because many of the factors and variables that affect the selection of these materials are not quantifiable or cannot be valued economically, the evaluation structure was divided into two steps.

The preliminary evaluation step considers the materials' properties, limitations, unique requirements, and prior experience relative to the objectives, characteristics, and needs of the planned project.

Those alternate surfacings that show a strong potential for being effective are subjected to an in-depth economic evaluation to determine the material with the least total present worth of life-cycle costs.

The total present worth of the life-cycle costs is selected to compare alternate surfacings. The benefit-cost ratio cannot be used because the benefits of the alternate surfacings, such as reduced operating costs, reduced maintenance costs, increased speeds, reduced travel time, increased safety, added comfort and convenience, energy savings, and environmental benefits of reduced air and water pollution, are not readily quantifiable. These factors are not expected to vary greatly from one surfacing to another.

The examples demonstrate the methodology and also show that typical low-volume forest roads can be more economically surfaced by using other materials than quality crushed aggregate. The high costs of quality aggregate and hauling can make locally available materials economically attractive. The sensitivity analysis demonstrates how much the decision can be affected by a few important variables and costs.

The ALTSURF computer program was developed to help decision makers perform the economic evaluation step. The use of ALTSURF with the preliminary evaluation step is a straightforward, valid procedure to evaluate the overall effectiveness and economic viability of alternate surfacings.

ACKNOWLEDGMENTS

The need for this study was identified at the national level of the Forest Service. The funds for the study were provided by the Washington office of the U.S. Forest Service. The work was performed under a contract with the San Dimas Equipment Development Center of the Forest Service. Their support is gratefully acknowledged.

Special thanks are extended to Ed Gililland (COR) of the Equipment Development Center for his assistance in all aspects of this study.

The contents of this paper reflect the views of the authors, who are solely responsible for the accuracy of the material presented. The contents do not necessarily reflect the official views of the sponsors.

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Roadway Management for Local Roads

DONALD M. WALKER AND PHILIP SCHERER

Local government managers responsible for low-volume roads in the United States are facing a dilemma. On the one hand, there is growing pressure to repair roads and provide an improved level of service. On the other hand, there is public pressure to reduce taxes. A roadway management system has been developed in response to the need for a better system to manage roadway budgets, maintenance, and selection of improvement projects. The system provides assistance in determining annual budgets, providing long-range planning, and selecting the improvements that are most cost-effective. It is simple enough to be understood and implemented by local officials with a limited technical background. Local values and goals are incorporated into the project selection process. An easy-to-use roadway surface condition rating scheme is an integral part of the management system. This evaluation tool allows local officials to review the overall condition of their roadways in light of their goals and future needs. The roadway management system has been successfully implemented by three local agencies in Wisconsin. Early results indicate that it has enabled local officials to better understand the need for improvements, establish objective priorities, and justify significant increases in the local road improvement budgets.

Local units of government responsible for maintaining the bulk of low-volume roads in the United States are faced with the following problems:

- Continuing pressure from individuals who want priority placed on fixing their roads;
- Continuing demands for more and better roads;
- Increasing expectations on the acceptable level of maintenance, even for low-volume, low-function roads;
- Increasing demands for accountability of the public dollar;
- Increasing need to get maximum benefit for every dollar of tax money collected;
- Continued inflation; and
- The problem of how to place priority on needed improvements given limited funds.

At the same time, they must balance the following problems:

- Continuing pressure to hold the line on property taxes;
- An almost certain reduction, if not total loss, of state and federal revenue-sharing;
- Little hope for any significant increase in state-administered transportation and highway aids; and
- The possible reduction or elimination of federal aid secondary programs and dollars.

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When pressured by these two strong and realistic forces, highway managers often find themselves responding to emergency needs; programming projects or improvements in response to political pressures; deferring timely maintenance and needed projects; experiencing an overall deterioration in the quality of highways; and using funds in an often less than efficient manner to respond to the most pressing projects or issues.

These problems exist at all levels of government. Towns, villages, cities, counties, and even states wrestle with these problems. However, many states, larger municipalities, and a limited number of counties have recently begun to develop some form of rational and objective method of programming, budgeting, and managing highways over an extended period of time. Smaller communities and those with a limited administrative staff have generally not had the time and resources to allow them to develop a comprehensive highway management program.

When local elected officials are faced with this dilemma, they commonly respond by trying to pressure state and federal agencies for more money. In reality, however, the evolving philosophy at both the state and federal level is that local units of government should bear the major responsibility for funding local road maintenance and improvement needs, particularly those needs that exist on low-volume roads.

Local officials should consider a number of options to solve these growing problems and pressures. For example, they can minimize their maintenance responsibilities by vacating existing roads, controlling new additions to the system, and establishing special assessments. They can also increase the efficiency of existing operations through equipment sharing, joint purchasing, and contracting out for certain tasks and projects. These are realistic and proven ways of improving efficiency.

Although many units of government have effectively initiated one or more of these measures, the growing disparity between roadway needs and fiscal resources is increasing almost universally.

In response to this need, a system has been developed to manage budgeting, maintenance, and roadway improvement projects for small communities in Wisconsin. The initial work was completed by the Northwest Wisconsin Regional Planning Commission (NWRPC) in 1984. The Wisconsin Transportation Information Center, a technology transfer center sponsored by the Federal Highway Administration, in cooperation with NWRPC, is providing training. The system had been adopted by a city, town, and county by mid-1986.

OBJECTIVES

The roadway management system has several prime objectives. It must first be helpful in determining annual budgets that meet community objectives. It must also be useful in providing long-range (2- to 5-year) planning for roadway improvement needs.

Although the concerns of local citizens and elected officials should be incorporated into the process, it is intended that poor investments that result from political pressure be minimized.

CHARACTERISTICS OF THE SYSTEM

Although many pavement management systems exist today, the unique characteristics of agencies that deal with low-volume roads must be considered. The following characteristics are considered important.

Simplicity

Local officials who have a limited technical background must be able to understand the procedures developed. In addition, it is important that the system not require the use of a computer initially. If the system can be explained in simple terms, and does not require any special equipment, two significant barriers to use by local officials will have been overcome. The use of a microcomputer is helpful, but a system that works on paper during the beginning stages can easily be converted to a microcomputer when and if the opportunity exists.

Current efforts are focused on the development of an initial roadway management system and the encouragement of its implementation. Although detailed analysis procedures such as pavement deflection, roughness, and economic optimization can also be useful to a low-volume road system, they are not needed immediately. The system can later be expanded to include these more sophisticated elements. In fact, it is anticipated that most local officials will actively seek improvements to and expansion of the system once its merits have been proven.

Local Implementation

Local elected officials and agency staff should be able to implement the system without extensive assistance from outside experts. Although some help is required during the initial development, the intent of this system is that most local highway officials will be able to use it independently in subsequent years.

Local Priorities

An effective system must incorporate the local community's issues and priorities. It will not be used in the future if local officials do not feel it is their system and reflects their values. The results must make sense to the officials in order for them to be accepted and implemented.

ROADWAY MANAGEMENT SYSTEM DESCRIPTION

The roadway management system for local roads includes the major steps involved in most other systems: inventory, classification, evaluation of roadway conditions, establishment of priorities, development of deficiency criteria, analysis, and recommendations of short- and long-range roadway improvements.

Inventory

The roadway inventory begins by creating short roadway segments that are similar in character. These segments vary in length; they can be several blocks in urban areas or between 1 and 3 miles in rural areas. The number of segments directly affects the volume of data that must be analyzed. Each segment must be reasonably consistent throughout its length in such important criteria as surface type, surface condition, construction history, and function. No segment should be longer than what would normally be undertaken as an improvement project. Although these guidelines appear vague, little difficulty has been experienced in determining reasonable roadway segment lengths.

Each segment needs an identification number and location. An identification number may be tied to an existing identification scheme or it may be arbitrary. It can also differentiate between different functional classifications of roadways.

The location of each roadway segment must be clearly identified. The use of roadway intersections is the most readily available and useful method of identification. Additional survey information, such as the section number, could also be useful but it is not essential.

Roadway mileage data are also required. Existing planning information can provide mileage to the nearest hundredth of a mile. If this information is not available, conventional automobile odometer mileage data can be used. However, estimates to the nearest five-hundredth of a mile should be made.

Other inventory information that is collected must be related to the data needs anticipated in the analysis phase. Therefore, it is suggested that inventory forms not be developed until the analysis is clearly defined. An example of an inventory form is shown in Figure 1.

Historical information on roadway maintenance and improvements should be included on the inventory form. Although this information is not likely to be used in detailed data analysis, it is extremely important for decisions at the project level. Some of this improvement history information can be collected in the office before the field inventory, but much information also can be collected during the field review. Site visits have a way of triggering the memory of local officials. Enough space should be left on the inventory form for comments because single ratings for a roadway segment often do not adequately reflect varying conditions. Comments can also be very helpful in project-level decisions.

Classification

A roadway classification scheme should be considered part of the roadway management system. The local road system may already have a functional classification system. However, previous functional classifications may not be sufficient for pavement management. Although almost all low-volume roads are classified as local in a functional statewide system, distinct differences may exist that indicate local priorities. The range of traffic volumes and functions is likely to be wide. Local officials are generally aware of these differences and can provide a refined roadway classification with little difficulty. In many cases, this classification is extremely important in ranking improvement needs.

A simple classification scheme may only involve identifying major and minor roadways. Local officials can add other

Town of Rutland
Highway Inventory Data

Inventory Date: ______
By: ______

SEGMENT & LOCATION

Road/Name: ______ Segment: ______
From: ______ To: ______
Length: ______ Section No. ______

USE & CLASSIFICATION	Ranking Points
Road Classification: ______	______
ADT: ______	
Access To: ______	
ROADWAY DATA	
Surface Type: ______	______
Surface Condition Rating: ______	______
Drainage Rating: ______	______
Comments ______	
GEOMETRICS	
Surface Width: ______	______
Shoulder Width: ______	______
Urban Roadway Width: ______	______
Horizontal Alignment Rating: ______	______
Vertical Alignment Rating: ______	______
R/W Width: ______	
Comments ______	
OTHER	
Comments ______	______
TOTAL RANKING POINTS	______

IMPROVEMENT HISTORY

Year: ______ ______ ______
Work Completed: ______ ______ ______
Estimated Cost: ______ ______ ______

FIGURE 1 Sample inventory form.

categories for areas that serve developed platted areas and dead-end roads, for example. Traffic counts would also be useful in this process. However, travel patterns and functions are generally well-known locally. Roads that are used by commuters for short-cuts and roads that serve local industry, schools, or emergency services are all easy to identify. These roads are likely to receive different priorities for improvement than those that serve a more limited purpose.

Roadway Condition

Condition data need to be collected on all segments of the roadway system. How much condition data are required is determined by the type of analysis and by the criteria established by local officials. This cannot be determined until final decisions on techniques for ranking local needs are completed.

Many techniques for assessing roadway surface condition are currently being used as agencies develop their own roadway management systems. A relatively simple surface condition rating system has been developed that worked well in its initial applications. Local officials can be trained in a relatively short time and actual data collection has presented few problems.

A copy of the rating system is shown in Figure 2. The judgments and evaluations that are normally used by roadway maintenance personnel are incorporated in the condition evaluation. It is usually apparent to the local roadway manager whether the surfaced road needs routine maintenance, seal-coating, overlaying, or complete reconstruction. This rating system reflects these judgments and is built on the existing capabilities of local road personnel.

The pavement surface condition rating is organized in the following manner. Ratings 9 and 10 indicate new or like new condition. Ratings of 7 or 8 indicate the need for routine

Surface Rating	Quantifiable Distress Measures	Generalized Surface Condition
Good 10	None	New plant mix
9	None	New cold mix or like new asphalt
8	a) No surface ravelling (loss of fines) b) Less than 100 feet/station of longitudinal cracks less than 1/2" in width c) 1-5 transverse cracks/station < 1/2" in width	Good surface for most of segment Can be maintained with routine maintenance and patching
7	a) b & c from above (8) b) Slight to no surface ravelling (loss of fine aggregate) c) Existing patches in good condition	
6	a) More longitudinal cracking, i.e. 300'/station crack width < 1/2" b) Transverse cracks 6-10/station and less than 1/2" in width c) Existing patches in fair condition d) Slight surface ravelling	Surface showing signs of ageing, but could experience extended life with timely seal coat and routine maintenance
5	a) a, b, & c from above (6) b) Moderate surface raveling c) First signs of block cracking, less than 25% of the area effected	
4	a) Longitudinal cracks now greater than 1/2" and < 200'/station b) 1 to 5 cracks/station and > 1/2" c) Block cracking between 25 and 50% of the surface area effected d) Existing patches in fair condition	Significant ageing and surface deterioration. Needs new mat or overlay. Beyond the point where a seal coat would be cost effective.
3	a) Longitudinal cracking more than 200'/station and > 1/2" b) Transverse cracking 6-10/station and > 1/2" c) Block cracking greater than 50% of the area d) Alligator cracking less than 25% of the area	
2	a) Alligator cracking more than 25% b) Transverse or longitudinal cracking banded c) Extensive patching, most in poor condition	Surface is totally deteriorated. New mat or overlay would not be adequate. Needs totally new road surface and improvements to base.
Poor 1	All of the above plus rutting greater than 1 inch	

FIGURE 2 Sample of pavement surface condition rating.

maintenance. Ratings of 5 and 6 indicate the need for preservation (seal coat). Ratings of 3 to 4 indicate candidates for overlay. Ratings of 1 to 2 indicate that complete reconstruction is necessary. Once the general category has been determined, the final rating is a matter of selecting the high or low value within that category. Quantifiable distress measures are helpful in making this selection.

This simple approach to surface rating makes it easy to identify maintenance and improvement projects. This information, together with priority ranking data, can then be used to provide short- and long-range budget needs.

There is a separate surface rating evaluation for gravel roads. A ranking of 1 to 5 is considered sufficient to describe the major categories of unsurfaced road conditions. Details of surface ratings for unsurfaced roads are provided in the following table. In this context, unsurfaced means unpaved. Unsurfaced roads in Wisconsin generally are gravel roads.

Surface Rating	*General Surface Condition*
5	Very sound road; good drainage, good base, and good gravel.
4	Fairly good road; good drainage, good base, but needs gravel.
3	Majority of road needs sand lift and gravel.
2	Little or no drainage; majority of road needs drainage work; needs sand lift and gravel; and width is possibly inadequate.
1	Little or no actual road profile or cross-section; needs almost total reconstruction, including drainage, base and sand lift, and gravel.

Other features must also be evaluated in the field. Horizontal alignment, vertical alignment, and drainage are important for rural projects. A set of rating schemes used in the Federal Highway Administration's Highway Performance Monitoring System was adopted. These schemes use fairly simple instructions and straightforward evaluations. They generally use word descriptions, such as poor, fair, good, and excellent. This rating system is further described in Table 1.

Priorities

A manager must consider a wide range of factors to determine the priority of roadway improvements. Most of the factors normally considered important by local officials are listed in Table 2. A ranking scheme can be developed by assigning weights to each of these factors. These weights are combined with the previously determined condition evaluation to develop individual improvement projects.

The commitment of local officials to using a roadway management system depends on the credibility of the system. Any ranking activity must generate projects that reflect local values and the common sense of the elected officials. Therefore, the initial determination of weighting factors and selection of ranking criteria must be made by the local officials.

The first three applications of this roadway management system evolved into three significantly different ranking schemes. For example, the county system developed criteria that were greatly disposed toward safety. The small city placed heavy emphasis on paving unsurfaced roads. The town developed

TABLE 1 ALIGNMENT AND DRAINAGE RATINGS

Rating	Description
Vertical Alignment	
Excellent	All grades (rate and length) and vertical curves meet minimum design standards appropriate for the terrain. Reduction in rate or length of grade would be unnecessary even if reconstruction is required to meet other deficiencies, such as capacity and horizontal alignment.
Good	Although some grades (rate and/or length) and vertical curves are below appropriate design standards for new construction, all grades and vertical curves provide sufficient sight distance for safe travel and do not substantially affect the speed of trucks.
Fair	Infrequent grades and vertical curves that impair sight distance and/or affect the speed of trucks if truck climbing lanes are not provided.
Poor	Frequent grades and vertical curves that impair sight distance and/or severely affect the speed of trucks and truck climbing lanes are not provided.
Horizontal Alignment	
Excellent	All curves meet appropriate design standards. Reduction of curvature would be unnecessary even if reconstruction is required to meet other deficiencies, such as capacity and vertical alignment.
Good	Although some curves are below appropriate design standards for new construction, all curves can be safely and comfortably negotiated at the prevailing speed limit on the section. The speed limit was not established by the design speed of curves.
Fair	Infrequent curves with design speeds less than the prevailing speed limit on the section. Infrequent curves may have reduced speed limits for safety purposes.
Poor	Several curves uncomfortable and/or unsafe when traveled at the prevailing speed limit on the section, or the speed limit on section is severely restricted because of the design speed of curves.
Drainage	
Good	Fully adequate drainage and cross-section design. No evidence of flooding, erosion, ponding, or other water damage.
Fair	Height of grade line, cross-section, or culvert capacity somewhat below the standard that would comply with standards if rebuilt. Drainage structures are structurally sound. Some added maintenance effort required because of drainage and sedimentation problems.
Poor	Evidence of severe flooding, ponding, erosion, or other drainage problems. Drainage structures may be in poor condition. Considerable excess maintenance effort required because of drainage and sedimentation problems.

TABLE 2 ROADWAY EVALUATION FACTORS

Criteria	Measures
Condition	
Surface condition	Surface rating (1-10), PSR, PCI, Asphalt Institute
Structural adequacy	Deflection tests, pavement layer thickness
Drainage adequacy	HPMS rating
Surface type	PCC, plant mix bituminous, road mix, seal coat, gravel, soil
Age	Years since last resurfacing
Geometrics	
Horizontal alignment	HPMS rating, number of curves per mile
Vertical alignment	HPMS rating, number of hills per mile
Lane width	Feet
Shoulder width	Feet
Passing opportunity	Percent passing
Functional	
Arterial	Major roadway—provides mobility
Collector	Minor roadway
Local	Provides property access—limited through traffic
Dead end	Serves one property—limited use
Service	
Ride	PSI, panel rating (1-5)
Safety	Accident rate, repeat accident location, may substitute geometrics for accident potential
Load limits	Posted bridges and roads
Parking	Designated space available
Right of way	Adequate to maintain drainage and vision
Snow drifting	Deep cuts, steep back slopes, brush
Spot problems	Various localized problems causing reduced level of service

criteria that favored preserving investments on higher function roadways. Although all three systems had many similar elements, the emphasis on particular elements varied greatly. This local sensitivity furthers local acceptance of the process and recommendations.

The factors that are used to set priorities generally fall into three or four broad categories: roadway condition, safety, function, and service. These categories were found to work well. Roadway condition relates to surface condition, structural adequacy, and drainage. A surface condition rating can be used with a drainage evaluation to cover this important category.

Safety analysis can present significant problems for low-volume roads. Detailed accident data are often unavailable. In addition, analysis techniques developed for high-volume roads may not be appropriate for low-volume roads. Surrogates for traffic accident data could be used, such as alignment, pavement geometrics, and identification of spot hazards. Intersections can also be evaluated separately if a significant problem exists.

Roadway function generally includes the previously mentioned functional classifications. Traffic count data could also be useful, if available. Service to local industry, schools, and commuter routes must be considered. The service category could include the general level of service provided and other spot conditions, such as ride quality, surface type, and snow drifting.

A scale of 0 to 100 is recommended for ranking improvements of individual segments. It has been found useful to use a low score to indicate a high level of need for improvement. An example of this approach is shown in Figure 3. Figures 4 and 5 are examples of earlier versions that used the opposite

A. PAVEMENT (55 Ranking Points)

Pavement Surface Condition Rating	Points
10	40
9	39
8	37
7	35
6	29
5	23
4	16
3	10
2	4
1	0

Gravel Surface Condition Rating	Points
5	40
4	20
3	15
2	10
1	0

Drainage Rating	Points
Excellent - 4	10
Good - 3	7
Fair - 2	3
Poor - 1	0

Surface Type	Points
Hot Mix Asphalt	0
Cold Mix Asphalt	3
Gravel	5

B. GEOMETRICS (25 Ranking Points)

Surface Width (Rural)	Points
22	10
20	7
18	4
16	0

Roadway Width (Urban)	Points
44	19
40	16
36	11
32	7
30	3

Shoulder Width (Rural)	Points
6	9
4	7
2	4
0	0

Horizontal (Curves)	Points
0-1 Curve/Mile	3
2 Curve/Mile	2
1 Turn, or 3 Curves/Mile	0

Vertical (Hills)	Points
No steep grades or hills	3
1 steep grade per mile	2
2 or more steep grades per mile	0

C. ROAD CLASSIFICATION (15 Ranking Points)

	Points
A - Major	0
B - Minor	5
C - Plat	10
D - Local	15

D. SPOT CONDITIONS (5 Ranking Points)

		Points
Safety	Frost Heave	0 - 5
Pot Holes	Rough Ride	
Bumps	Snow Drifting	
Poor Culvert	Narrow Right of Way	

FIGURE 3 Sample of Town of Rutland ranking and deficiency criteria.

SEGMENT No. ______

			POSSIBLE POINTS	ITEM SCORE
A. PAVEMENT (21 points maximum)				
1) Age (2 points maximum)				
• Over 20 years			2	
• 10 to 20 years			1	
• 0 to 9 years			No points	______
2) Pavement Condition (11 points maximum)				
Rating	Points	Rating		
• 1	11	• 6	6	
• 2	10	• 7	5	
• 3	9	• 8	4	
• 4	8	• 9	No points	
• 5	7	• 10	No points	______
3) Pavement Width (8 points maximum)				
• Less than 20 ft.			8	
• Less than 22 ft.			4	______
B. SAFETY (25 points maximum total - can be combined)				
Accident Record (Within last three years)				
• 1 or more preventable death accidents			10	
• 2 or more preventable personal injury accidents			10	
• 2 or more preventable property damage accidents			5	______
C. GEOMETRICS (31 points maximum total)				
1) Horizontal Alignment (13 points maximum) (Posted curves per Mile)				
• Three or more occurrences			13	
• Two or more occurrences			8	
• One occurrence			3	______
2) Vertical Alignment (13 points maximum) (Steep grades Per Mile)				
• Three or more occurrences			13	
• Two or more occurrences			8	
• One occurrence			3	______
3) Shoulder Width (5 points maximum)				
• No shoulder			5	
• Less than 2 ft. shoulder			3	
• Shoulder 2 ft. to 4 ft.			1	______
D. TRAFFIC VOLUMES (16 points maximum)				
• Over 2000 ADT			16	
• 1500 - 1999 ADT			13	
• 1000 - 1499 ADT			10	
• 500 - 999 ADT			7	
• 250 - 499 ADT			4	
• Less than 250 ADT			No points	______
E. FUNCTIONAL CLASSIFICATION (7 points maximum)				
• Arterial			7	
• Major Collector			5	
• Minor Collector			3	
• Local Road			No points	______
TOTAL SCORE:				______

FIGURE 4 Sample of Washburn County segment score sheet.

approach. It is helpful to first distribute the 100 points to each major category, and then weight individual factors within the category.

Some adjustment of the overall ranking is likely to be necessary after the early analysis. These adjustments should not be considered a weakness in the system, but a necessary result of the subjective nature of the ranking process.

Deficiency Criteria

An evaluation of existing conditions necessitates a comparison with desirable or acceptable conditions. Geometric standards usually exist for surface width, shoulder width, and maximum grades. Alignment and drainage can be compared to excellent condition descriptions in Table 1. Other factors are more subjective. Experience with local officials has indicated that they have little difficulty in establishing relative evaluation criteria. Examples of specific criteria are shown in Figures 3, 4, and 5.

Analysis and Recommendations

The identification of roadway improvements could be either a simple or a complex process. A simple approach, which is useful

Date ____________

Segment Name: ____________
Segment No.: ____________

	Possible Points	Segment Points
A) FUNCTIONAL CLASSIFICATION (30 points maximum)		
1) City Classification C	30	
2) City Classification D	15	
3) City Classification E	0	______
B) ROADWAY DATA (65 points maximum)		
1) Surface Width		
• Less than 75% of Standard	5	
• 76 to 99% of Standard	2.5	
• 100% or more of Standard	0	______
2) Travel Lanes		
• Less than 75% of Standard	5	
• 76 to 99% of Standard	2.5	
• 100% or more of Standard	0	______
3) Parking		
• Both sides	2	
• One side	1	
• No parking	0	______

4) Surface Type

City Class	Bitum	Gravel
C	10	20
D	5	15
E	0	0

5) Surface Condition

Bituminous	Gravel
1-25	1-25
2-22	2-20
3-19	3-15
4-16	4-10
5-13	5-5
6-10	
7- 7	
8- 4	
9- 1	
10- 0	

______

6) Drainage
8 - Erosion problems
4 - Need ditching
2 - Needs curb and gutter
0 - Satisfactory ______

C) MISCELLANEOUS DEFICIENCIES (5 points maximum) ______
TOTAL SEGMENT SCORE ______

FIGURE 5 Sample of City of Washburn segment score sheet.

on a small roadway system, is to list all segments that fall into each of three surface rating categories. Roadway segments rated 5 or 6 are candidates for preservation (seal coats); those rated 3 and 4 need resurfacing; and those rated 1 and 2 need reconstruction. A review of this list allows the manager or local official to select projects directly. Annual work programs can be developed using average cost estimates for each category of work (seal-coating—\$5,000/mi, resurfacing—\$40,000/mi, reconstruction—\$100,000/mi). This approach may be similar to existing procedures, but it has the advantage of documenting existing conditions. In future years the lists will show if more projects are being added to the needs list than are being completed. This basic approach could be useful in starting a roadway management system.

A more detailed analysis involves a comparison of existing conditions with the deficiency criteria. The weight assigned to each deficiency is then used to rank each roadway segment. Specific improvement projects will then evolve from the list of segments in order of rank.

It is important that a description of the overall current condition of the roadway system be produced. This can be done in graph form; examples are shown in Figures 6 and 7. Local officials will be better able to determine and justify adequate budget requirements with this information. This information is also valuable in assessing trends. Changes in overall roadway conditions may indicate progress or deterioration. This information is also useful in setting future budget levels.

Probably the most difficult task in this process is that of developing a general budget range that can be maintained over a reasonable period of time. The overall objective of the entire process is to develop budgets that are relatively consistent and to avoid budgeting and management by crisis. In many if not most cases, municipal budgets are based on a percentage increase or decrease over the past year's budget. The process described in this document is intended to establish a budget that is primarily based on documented needs.

The analysis described earlier identifies roadway segments in need of improvement. The size of an annual budget will depend

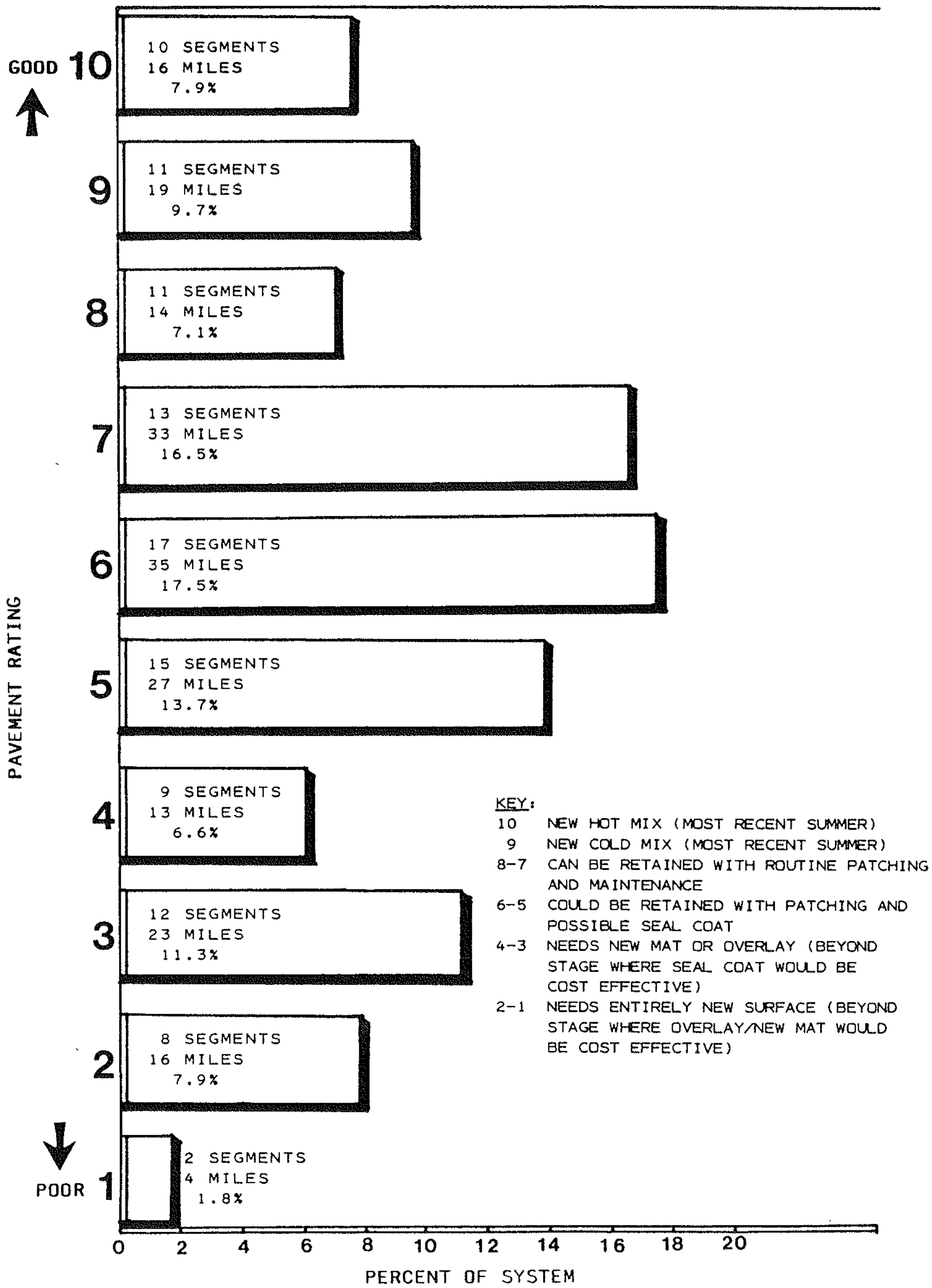

FIGURE 6 Washburn County pavement conditions.

on how fast this need is met and the rate of future roadway deterioration. Initial budget estimates can be developed by trying to complete all currently identified needs in 3 to 5 years. A review of the local agency's goals will determine if this approach is adequate and feasible.

It has been estimated that it is necessary to budget between $1,500 and $2,500 per year per mile of paved roadway in Wisconsin. This estimate is for improvements only and does not include routine maintenance costs such as crack sealing, patching, mowing, and snow removal.

It is also important for communities to compare their budget expenditures with those of other, similarly sized communities. This puts budgeting needs in perspective and is often helpful in justifying appropriate expenditures. However, comparisons should be made with communities that are meeting their goals and have a desirable roadway system.

IMPLEMENTATION

Most local highway agencies require some technical assistance in implementing a roadway management system. Because most agencies have a limited technical staff, the need for assistance can be great. Experience has shown that persons from a wide range of sources can be assembled to provide this technical assistance. Technical staff from universities, regional planning

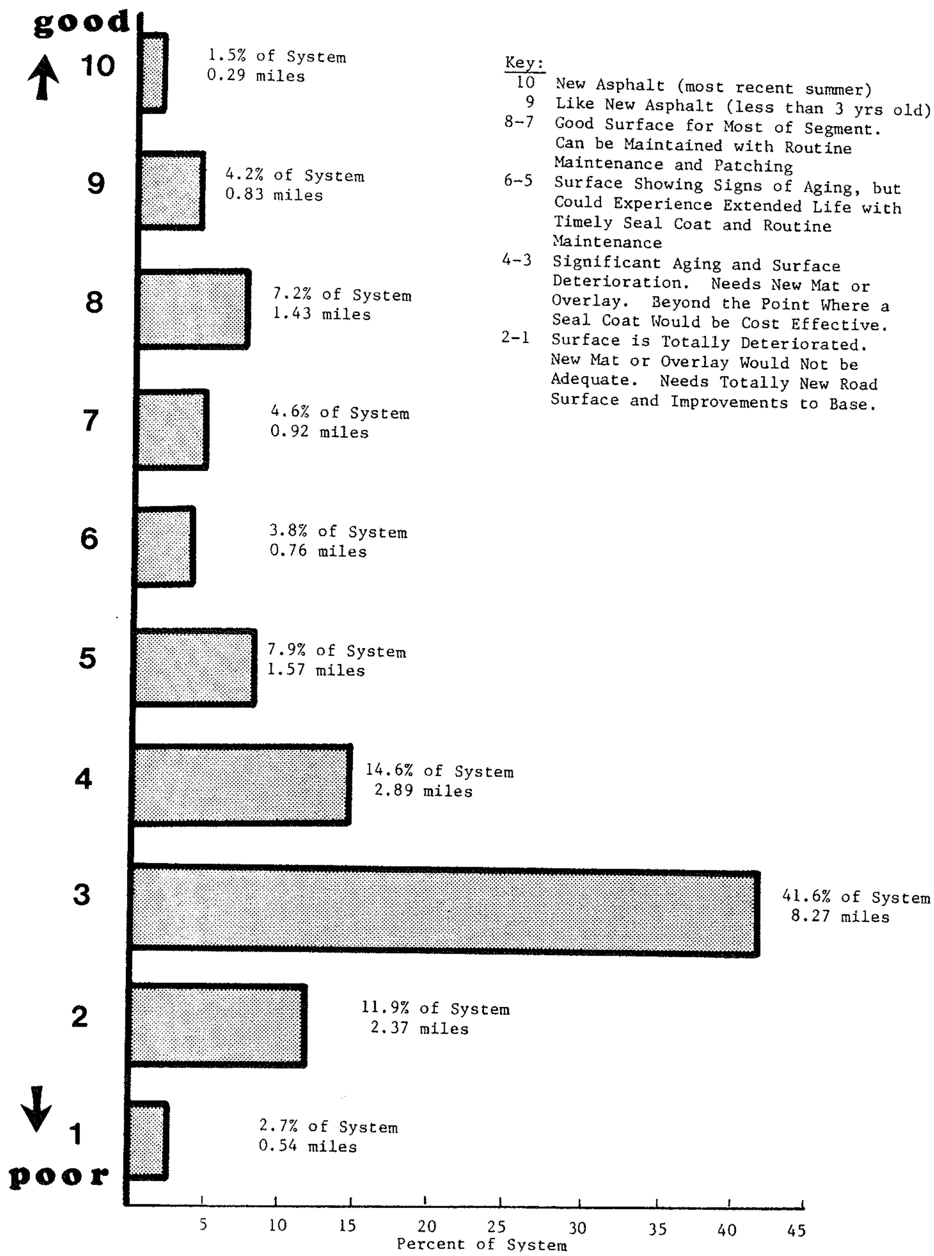

FIGURE 7 City of Washburn street conditions.

commissions, state transportation departments, larger local agencies, and consulting firms have performed effectively as project leaders.

The typical activities required for implementing a roadway management system are shown in the flow chart in Figure 8. The process usually requires four to six meetings with local agencies. The initial meeting is important to establish the scope of the study and responsibilities. The roadway system should also be discussed at this meeting. A roadway classification system should then be started. Through the discussion of the roadway system and its different functions, the project leader can begin to develop not only a roadway classification scheme, but some insights into local priorities and goals. Local priorities should be reflected in final decisions, not the bias of those providing technical assistance.

Deficiency criteria and rating systems will evolve at subsequent meetings. It is helpful to provide a range of criteria from which local officials can review and select. A useful starting point is provided in Table 2. Each local agency is likely to focus on different elements or emphasize different issues. They should, however, be appraised of generally accepted standards.

Once the criteria are established, the technical assistance team will help collect and analyze field condition data. A slide series was successfully used to train local officials in how to rate pavement surface conditions. Examples of pavements in each category should first be presented. Then other roadway examples should be shown and rated. Consistent ratings can be achieved in a 1-hour training session.

The extent of the data analysis required obviously varies with the complexity of the system. The analysis primarily consists of tabulating data and assembling the output into a format that can be understood by local officials. Experience has shown that complete implementation normally requires six 2-hour meetings. Data analysis has been in the range of 10 to 20 hours. The

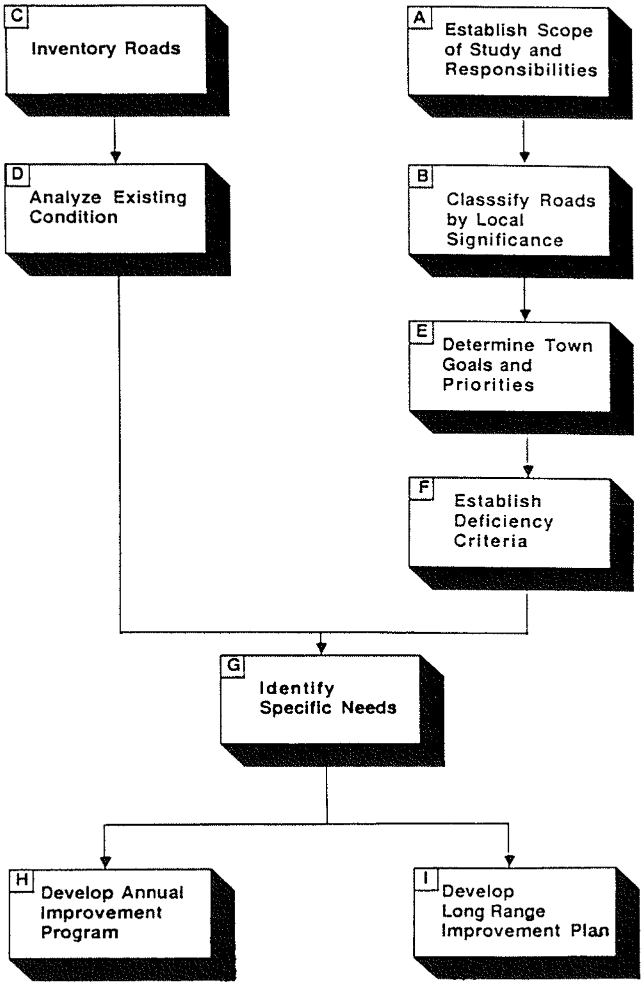

FIGURE 8 Town of Rutland road management study outline.

collection of field data on condition assessment can vary significantly. However, experience has shown that about 1 to 2 days of field data collection are required for every 100 mi of road inventoried.

RESULTS

The roadway management system was fully implemented by the three agencies by mid-1986. The process was accepted by local elected officials and is being used to manage and budget roadway improvements. The agencies are providing the time and needed staff to continue its use.

The benefits cited by the local officials center on an improved understanding of the roadway improvement needs, objective determination of priorities, and additional commitments toward meeting those needs. For example, the city's 1986 street budget increased threefold and the town's increased fourfold.

CONCLUSIONS

The following conclusions can be drawn from experience in the development and implementation of the roadway management system for local communities in Wisconsin.

- A significant need exists for the use of a roadway management system by local governments. The sincere and enthusiastic interest and support of local governments clearly demonstrates this need. Requests for assistance in implementing a similar system currently exceed the ability to provide it.
- The basic system was able to meet the needs of a variety of local agencies in Wisconsin, including a town, city, and county. The system has been used to develop roadway improvement programs and has been accepted by local elected officials. Although a more sophisticated system is useful, a basic approach has been successful in increasing the interest of local officials in a roadway management system. Refinement and improvement of the system are possible after the initial system has been accepted.
- A simplified pavement surface condition rating system has been developed. It uses the judgment and experience of existing maintenance personnel, and provides an organized and understandable way of assessing current conditions. The surface condition rating can be used alone or in conjunction with a more comprehensive management system.
- The development of future budgets and priorities is relatively easy once the existing conditions are described and understood. Of equal importance is the fact that budgets and priorities are based on objective criteria and standards that were combined with roadway condition data.
- A significant technical assistance effort is required to implement this system with local agencies. A wide variety of technical personnel has been successful in providing this assistance.

ACKNOWLEDGMENT

Financial support for this study was provided by the NWRPC, the Federal Highway Administration, and the involved local agencies. The views and findings expressed in this paper are those of the authors and do not necessarily reflect those of the NWRPC, FHWA, University of Wisconsin, or local agencies.

A Method for Rating Unsurfaced Roads

R. A. EATON, S. GERARD, AND R. S. DATTILO

A method for rating the surface condition and drainage of unpaved 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. Types of distress found in unpaved roads are categorized and listed in the manual. For each type of distress listed there is a description of the type and the level of severity, an illustration, and a measurement method. The manual also includes instructions on how to inspect unsurfaced road conditions, a field inspection worksheet, and a family of deduct-value curves for the distress types and associated severity levels. The curves were validated using data gathered during seven field surveys throughout the United States. The surface and drainage rating method and maintenance management strategies can be used alone, or they can be adapted for use with any existing computerized pavement management system (PMS). The rating method and strategies are compatible with the PAVER PMS developed by the U.S. Army Corps of Engineers and the American Public Works Association. With appropriate software modifications, an unsurfaced roads component of the PAVER PMS will be available for use to provide local highway agencies with a more comprehensive roadway management system.

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. These PMSs cannot currently be used for unsurfaced roads. An unsurfaced road component that can stand alone or be used with any of these PMSs would provide local highway agencies with a comprehensive roadway management system that would be 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. Only Phases I and II are addressed in this paper.

Phase I consisted of the development of a field manual for rating the condition of unsurfaced low-volume roads. Maintenance management practices employed by townships, the military, and municipal, county, and state governments were used to develop this rating system. The effort also focused on reviewing past and current maintenance practices, and identifying and conducting field surveys of unsurfaced road distress types.

Phase II consisted of a series of field surveys that were directed at validating the field manual. These surveys provided the information required to define and describe the distress types and their associated severity levels. In addition the surveys provided the data needed to develop the deduct-values associated with each distress and severity level. This phase was conducted through the cooperative efforts of the Federal Highway Administration (FHWA); U.S. Army Corps of Engineers Cold Regions Research and Engineering Laboratory, Construction Engineering Research Laboratory, and Waterways Experiment Station; U.S. Army Forces Command; U.S. Army Training Command; U.S. Army Facilities Engineering Support Agency; U.S. Army Engineer District, Tulsa; Town of Hanover, New Hampshire; State of New Hampshire Department of Transportation; Hardin County, Kentucky; Long County, Georgia; Ft. Knox, Kentucky; Ft. Stewart, Georgia; Ft. Chaffee, Arkansas; Ft. Irwin, California; Ft. Lewis, Washington; and State of Alaska Department of Transportation. The CRREL Special Report "Unsurfaced Road Distress Measurement Field Manual" provides more detail about Phase I.

PROJECT BACKGROUND

The Phase I development of the unsurfaced roads field condition rating method and manual was funded through the FHWA Rural Technical Assistance Progam (RTAP). The research study work has been conducted as "RTAP Project No. 29: Revising the PAVER Pavement Management System for Use on Unpaved Roads." A concise description of the RTAP program and how the study originated follows.

The U.S. Congress appropriated funding for RTAP beginning in 1982. The program is focused on roads, bridges, and public transportation in rural areas. It is mainly aimed toward county, municipal, and local agency personnel. Under the program, several RTAP centers were established, primarily at institutions of higher learning. Through these centers, local agency training is completed and transfer of new technology by various other means is also accomplished.

The PAVER PMS was developed by an unsurfaced roads component of the U.S. Army Corps of Engineers (COE) and the American Public Works Association (APWA). It was suggested to FHWA by the Vermont Local Roads Program (VLRP), which is the RTAP center at St. Michael's College in Winooski, Vermont. The VLRP RTAP center had been assisting local agencies with implementation of the COE-APWA PAVER PMS, and found that it, as well as others, did not include provisions for unpaved roads. Several local agencies indicated that having an unpaved roads component in the PAVER system would be helpful because this category of roads constitutes a major portion of the roadway system they were responsible for maintaining. They also agreed to work on a

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project to develop this component. Based on the VLRP suggestion and evidence of local agency need and support for such a project, the Phase I effort to develop a method and manual for rating the field condition of unpaved roads was initiated in cooperation with the COE. To ensure compatibility of the developed method and manual with the COE-APWA PAVER PMS, the COE Construction Engineering Research Laboratory (CERL), which originally developed the PAVER system, was also asked to participate in the study. A prototype rating method and manual were developed, and the need to validate them under actual field conditions was recognized.

As a result of the Phase I study, the Phase II field validation effort was approved and initiated. The Phase II work was jointly funded through an extension of RTAP Project No. 29 and by contributions from several U.S. Army agencies. An executive steering committee was formed by the principal funding agencies to coordinate the work activities in Phase II. This committee included representatives from the U.S. Army's Facilities Engineering Support Agency, Forces Command, Office of the Chief of Engineers, Training Command, and Corps of Engineers research laboratories (CERL, CRREL, and WES), and the Federal Highway Administration. A representative of the Vermont Local Roads Program, RTAP Center, was also a liaison member. The actual field validation was performed at military installations and nearby areas. The selected sites represented the varying unpaved road soil and surface aggregate conditions, environmental conditions, and degrees of maintenance provided. The sites were located in Kentucky, Georgia, Oklahoma, Arkansas, New Hampshire, Vermont, California, Washington, and Alaska. Field condition rating panels consisting of representatives of the military installations and local areas have used the Phase I prototype method and manual to ensure that the unpaved road distress types, severity levels, and deduct-value curves are accurate and repeatable. A final field manual, including final curves, is being published as the CRREL Special Report mentioned earlier. The COE Army Technical Manual 5-623, *Pavement Maintenance Management*, published in 1982, will be modified to include unsurfaced roads.

STATEMENTS AND DEFINITIONS

The following are the statements and definitions used in the development of the manual:

- *Pavement Management*: Differences exist between paved roads and unpaved or gravel roads. This is primarily because of the short life span of gravel roads compared to paved roads. Long-term planning for a paved road would be 5 to 15 years, whereas for a gravel road it would be 1 to 2 years.
- *Unsurfaced Road Management*: An unsurfaced road is any road that does not have Portland cement concrete, asphalt concrete, or any other surface treatment. The normal maintenance of unsurfaced roads consists of blading with a road grader. Unsurfaced road management is based on a dynamic situation in which road conditions change significantly between one grading or blading and the next. Blading or grading should be conducted three or four times a year, and planning or scheduling should be done on an annual basis.
- *Distress*: Distress signifies any undesirable condition of an unsurfaced road. Use of this term maintains compatibility with PAVER.
- *Roughness*: This term refers to the ride quality of an unsurfaced road.
- *Unsurfaced Road Condition Index (URCI)*: This index is a numerical indicator, based on a scale of 0 to 100, that measures the road's operational condition; it corresponds to the PCI (pavement condition index) in the PAVER management system.
- *Inspection*: As used in this manual, "windshield" inspections consist of driving the full length of an unsurfaced road at 25 mph (the speed may be higher or lower depending on road conditions or local practice) in a pickup truck to determine the overall surface and drainage conditions four times a year (once each season). Relative surface condition ratings and drainage problems can be noted for all unsurfaced roads within the military installation, town, county, or city limits. General estimates of maintenance needs and priorities can be made from this initial inspection.
- *Measurements*: Measurements are the collection of detailed data on the roadway's surface and drainage conditions by highway personnel. After the initial inspection ride, a representative 100-ft section of road is selected in which actual measurements of distresses are taken. The measurements are needed to develop the numbers for the URCI. The section should be permanently marked so that future measurements will be taken in exactly the same location.
- *Deduct-Values*: As used in PAVER, the deduct-value is a number from 0 to 100, in which 0 indicates that a particular distress has no impact on road conditions, and 100 indicates an extremely serious distress that causes the road to fail. Deduct-values for each distress and severity level are presented in this paper.
- *Delphi Panel*: A Delphi panel is a group of experts on a subject who are brought together to discuss and document an area of concern.

FIELD MANUAL DEVELOPMENT

The field condition rating manual was developed by accomplishing the following tasks:

- Conducting an extensive literature search on the design, construction, operation, and maintenance of unsurfaced roads;
- Convening a series of workshops using the Delphi panel technique, in which the panel is predominantly composed of unsurfaced road experts from New England;
- Conducting discussions with local, state, federal, and university personnel; and
- Conducting a number of on-site field trips to survey unsurfaced road distress problems, how these problems manifest themselves, and what maintenance strategies are used to combat them.

First, an extensive review was conducted of available published information on operations and maintenance practices, maintenance management systems, construction and design, and traffic volumes and loads of unsurfaced low-volume roads. The literature search included a thorough review of documents, reports, manuals, and fact sheets prepared by a wide spectrum of organizations, including the Transportation Research Board, the Federal Highway Administration, the U.S. Department of Agriculture Forest Service, the U.S. Army Corps of Engineers

Construction Engineering Research Laboratory, the New Hampshire Department of Transportation, the U.S. Army Facilities Engineering Support Agency, the American Public Works Association, and the Vermont Local Roads Program. These documents provided a good background for the Delphi panel workshops and ensured that the rating and maintenance management system developed for unsurfaced roads was compatible with existing methods, procedures, and systems. Based on this review, it became apparent that this effort was not duplicating previous or ongoing efforts, and that it was worthwhile.

Three Delphi panel workshops were held with New Hampshire and Vermont local and state highway agency personnel. The purpose of the workshops was to prepare a preliminary draft of the distress rating and identification manual to be used in the Phase II field validation work. The Delphi technical panel accomplished this task, and the results of its efforts were documented in an interim, unpublished project report. In addition, the panel brought up and discussed many other topics related to construction, operation, and maintenance of unsurfaced roads. These three workshops provided the major contributions to the manual. They provided the background information on how unsurfaced roads are currently being maintained; identified and categorized the unsurfaced road distresses; identified some economic, political, and social problems; and outlined the information that should be in the manual.

Another major goal of the workshops was to present the information so that it could be readily understood by highway personnel and help them maintain their roads and conduct their budget reviews.

UNSURFACED ROAD DISTRESSES

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, the manual was modified by combining the two indices so that it currently lists the following seven distresses:

- Improper cross-section,
- Roadside drainage,
- Corrugations,
- Dust,
- Potholes,
- Rutting, and
- Loose aggregate.

Each of the following sections is structured to provide a description of the type of distress, definitions of its severity levels, and instructions on how to measure both the distress and its severity level. The accompanying figures depict the deduct-curves for each of the seven distresses.

Improper Cross-Section

Description

Improper cross-section is the result of the road surface not being properly shaped or maintained to carry water to the ditches. This condition is evidenced by water ponding on the road surface, water draining or running along the road surface, lack of a crown on the road, or road surface erosion caused by water runoff.

Severity Levels

L—Small amounts or evidence of ponding water on the road surface or a completely flat road surface (no cross-slope), or both.
M—Moderate amounts or evidence of ponding water on the road surface or a bowl-shaped road surface, or both.
H—Large amounts or evidence of ponding water on the road surface or severe depressions in the wheel paths on the road surface, or both.

Measurement

Improper cross-section is measured in linear ft per 100-ft section from outside shoulder break to outside shoulder break. Different severity levels can exist within the 100-ft sample unit. A maximum of 100 linear ft can be measured.

The deduct-values are shown in Figure 1.

Roadside Drainage

Description

Poor drainage causes water to pond. Drainage problems occur when ditches and culverts are not in the proper condition to adequately direct and carry runoff water. This condition is evidenced by overgrown or debris-filled ditches, ditches that have not been properly shaped or maintained, water running across or down the road, and areas in which the ditches have begun to erode into the roadway.

Severity Levels

L—Small amounts of:
- Ponding water or evidence of ponding water in ditch, and
- Overgrowth or debris in ditch.

M—Moderate amounts of:
- Ponding water or evidence of ponding water in ditch,
- Overgrowth and debris in ditch, and
- Evidence of erosion of ditch into shoulder or roadway.

H—Large amounts of:
- Ponding water or evidence of ponding water in ditch,
- Water running across or down road,
- Overgrowth and debris in ditch, and
- Erosion of ditch into shoulder or roadway.

Measurement

Drainage problems are measured in linear ft per 100-ft section parallel to the road centerline, from the outside shoulder break perpendicular to and away from the road. It is possible to have a maximum of 200 linear ft of roadside drainage distress.

The deduct-values are shown in Figure 2.

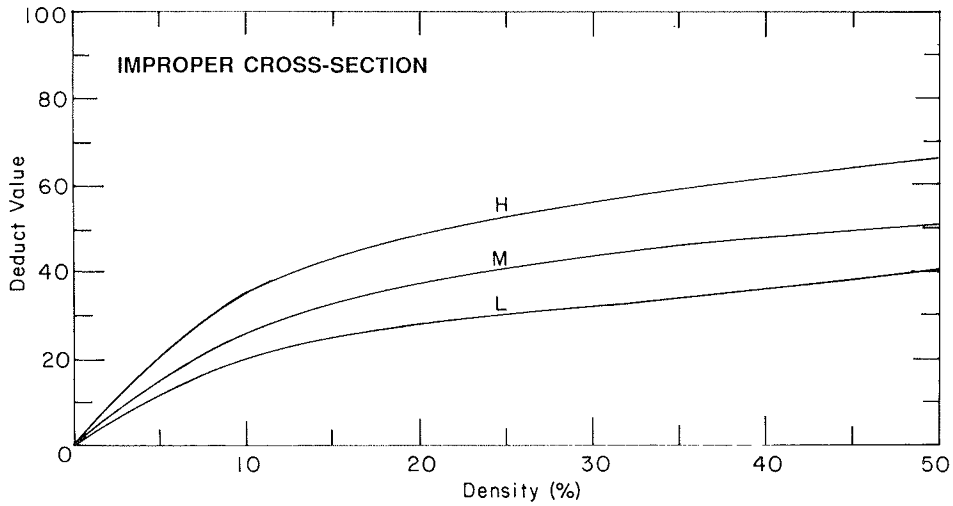


FIGURE 1 Deduct-value curves for improper cross-section.

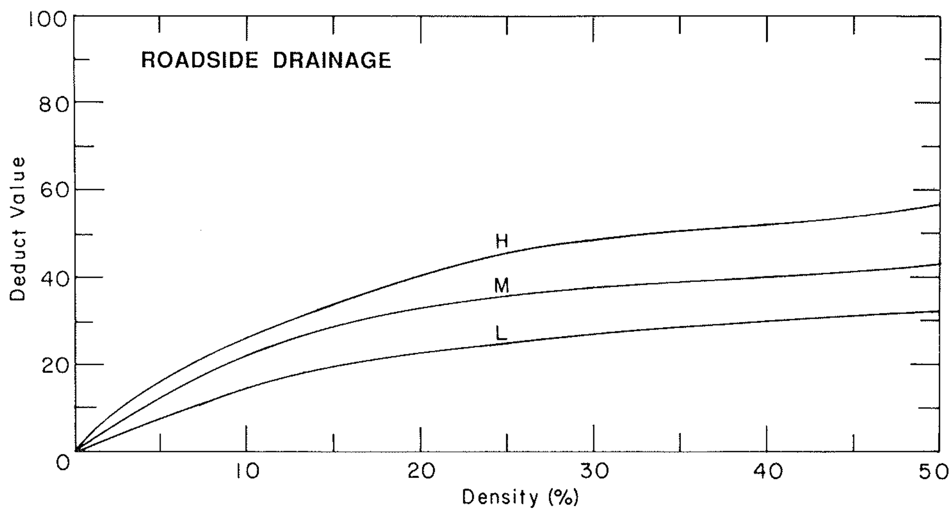


FIGURE 2 Deduct-value curves for roadside drainage.

Corrugations

Description

Corrugation, also known as washboarding, is a series of closely spaced ridges and valleys or ripples that occur at fairly regular intervals. The ridges are perpendicular to the traffic direction. This type of distress is usually caused by traffic action and loose aggregate. These ridges usually form on grades or curves, in areas of acceleration or deceleration, or in areas in which the road is soft or potholed.

Severity Levels

L—Corrugations less than 1 in deep or low-severity roughness, or both.
M—Corrugations 1 to 3 in deep or medium-severity roughness, or both.
H—Corrugations deeper than 3 in or high-severity roughness, or both.

Measurement

Corrugation is measured in square feet of surface area per 100-ft-long section. It must not exceed the total area of the 100-ft-long section.

The deduct-values are shown in Figure 3.

Dust

Description

The abrasive action of traffic on unsurfaced roads eventually loosens the larger aggregate particles from the soil binder. As traffic passes, dust clouds create a danger to trailing or passing vehicles and cause significant environmental problems.

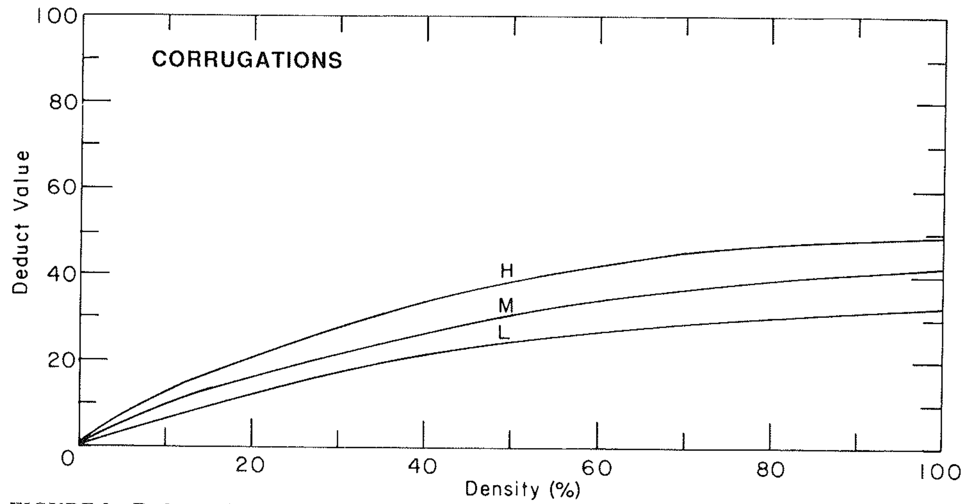


FIGURE 3 Deduct-value curves for corrugations.

Severity Levels

Normal traffic produces the following levels of severity:

L—Thin dust that does not obstruct visibility,
M—A moderately thick cloud that partially obstructs visibility and causes traffic to slow down, or
H—A very thick cloud that severely obstructs visibility and causes traffic to significantly slow down or stop.

Measurement

Dust is measured by driving a vehicle at 25 mph and observing the dust cloud; the dust is estimated to be thin, moderately thick, or very thick.

Dust is not rated by density. The severity of the distress is determined by the size of the dust cloud generated by traffic and the reduction in visibility caused by the dust.

The deduct-values for the levels of severity are as follows:

Low	2 points
Medium	5 points
High	15 points

Potholes

Description

Potholes are small, bowl-shaped depressions in the road surface that are usually less than 3 ft in diameter. Their growth is accelerated by free moisture collection inside the hole. Potholes are produced when traffic abrades small pieces of the road surface. The road then continues to disintegrate because of loosening surface material or weak spots in the base or subgrade.

Severity Levels

The levels of severity for potholes under 3 ft in diameter are based on both the diameter and the depth of the pothole according to the following table:

	Average Diameter			
Maximum Depth	*< 1 ft*	*1-2 ft*	*2-3 ft*	*> 3 ft*
1/2-2 in	L	L	M	M
2-4 in	L	M	H	H
4 in +	M	H	H	H

If the pothole is over 3 ft in diameter, the area should be determined in square feet and divided by 5 ft^2 to find the equivalent number of holes.

Measurement

Potholes are measured by counting the number that are of low, medium, and high severity in a 100-ft-long section and recording them separately by severity level.

The deduct-values are shown in Figure 4.

Rutting

Description

A rut is a surface depression in the wheel path. Rutting is caused by a permanent deformation in any of the road layers or subgrade. It results from repeated traffic loads, especially when the road is soft. Significant rutting can lead to major structural failure of the road.

Severity Levels

L—Ruts less than 1 in deep or low-severity roughness, or both.
M—Ruts 1 to 3 in deep or medium-severity roughness, or both.
H—Ruts deeper than 3 in or high-severity roughness, or both.

Measurement

Rutting is measured in square feet of surface area in a 100-ft-long section. The total square feet of rutting must not exceed the total area of the 100-ft-long section.

The deduct-values are shown in Figure 5.

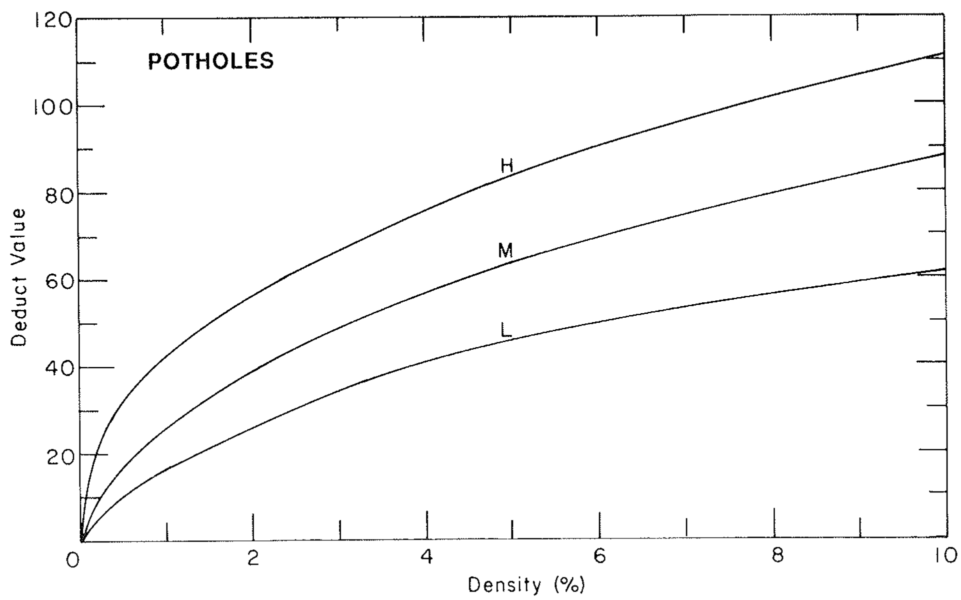


FIGURE 4 Deduct-value curves for potholes.

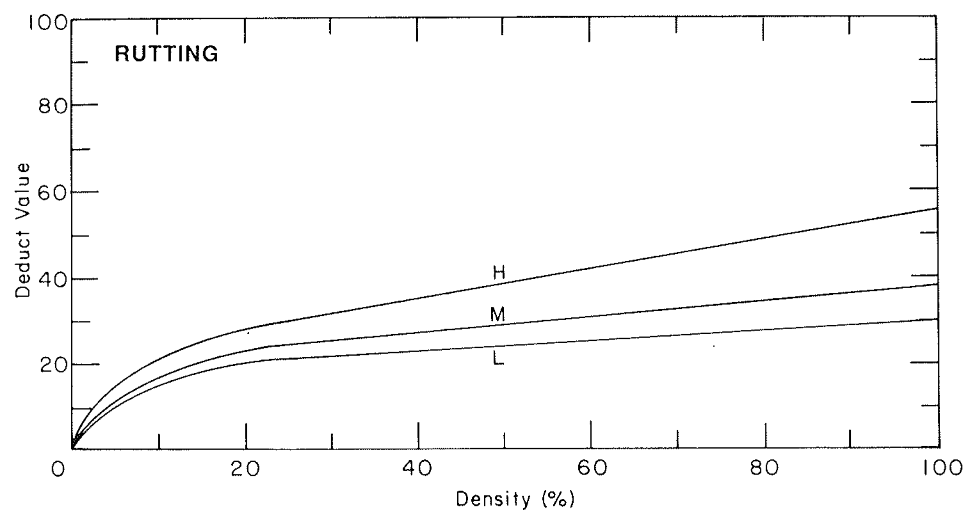


FIGURE 5 Deduct-value curves for rutting.

Loose Aggregate

Description

The abrasive action of traffic on unsurfaced roads eventually loosens the larger aggregate particles from the soil binder. This leads to base aggregate particles on the road surface or shoulder of the road. Traffic moves loose aggregate particles away from the normal road wheel path and forms berms in the center or along the shoulder of the roadway or less-traveled area, parallel to the road centerline.

Severity Levels

L—Loose aggregate on the road surface or an aggregate berm on the shoulder or less-traveled roadway area of less than 2 in, or both.
M—Moderate (2 to 4 in) aggregate berm on shoulder or less-traveled roadway area; excessive fines are usually found on the roadway surface.
H—Large (greater than 4 in) aggregate berm on shoulder or less-traveled roadway area.

Measurement

Loose aggregate is measured in linear ft in a 100-ft-long section parallel to the road centerline.

The deduct-values are shown in Figure 6.

FIELD VALIDATION SURVEYS

Based on the workshops, the relative importance of each type of distress was established. This information was used to develop

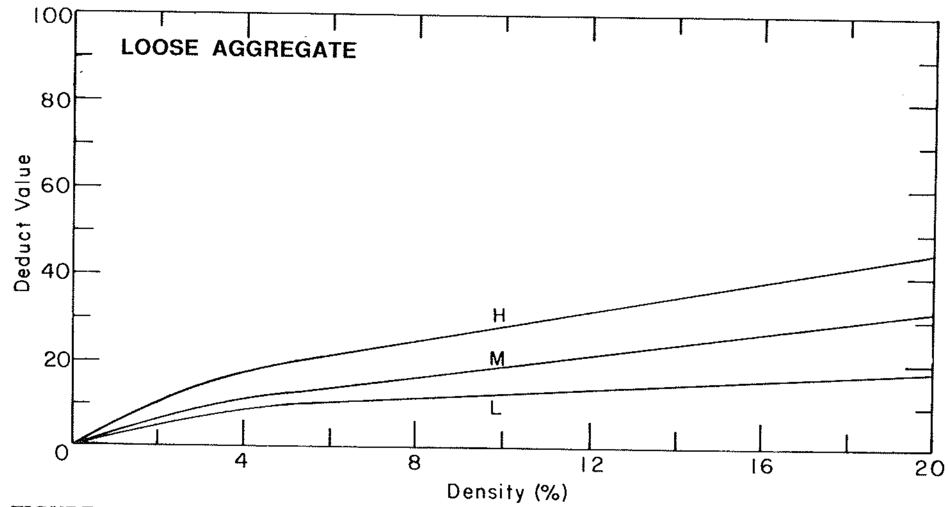


FIGURE 6 Deduct-value curves for loose aggregate.

the deduct-values for each type of distress and the associated severity levels for the initial field validation survey at Ft. Knox and Hardin County, Kentucky.

The initial field validation survey team was composed of eight members. Twelve 100-ft road sections were selected for evaluation and measurement. Each member of the team rated the section according to the Unsurfaced Road Condition Index (URCI) as follows:

0-25	Poor
25-50	Fair
50-75	Good
75-100	Excellent

Field evaluation surveys were then conducted at Ft. Stewart and Long County, Georgia; southeastern Oklahoma; Ft. Chaffee and northwestern Arkansas; central New Hampshire and Vermont; Ft. Irwin and southern California; Ft. Lewis, Washington; and central Alaska from Anchorage to Prudhoe Bay.

In addition to the original URCI estimate, each team member was required to assess the distresses in each section. These distresses were used to compute ratings based on the deduct-values. Based on both the ratings and the distress measurements, new adjusted deduct-values were developed that resulted in the least difference between the computed and estimated ratings.

These adjusted deduct-values were used to compute ratings at subsequent field validation surveys. The mean difference between the estimated and the computed ratings was -0.1 point. The average dispersion of any road section among the team members was approximately 6.0 points. These differences in the estimated and computed ratings are extremely small considering the significant differences in the locations, soil conditions, composition of the survey teams, and road conditions.

A sample of the inspection sheets that were developed to conduct the unsurfaced road measurement surveys is shown in Figure 7.

UNSURFACED ROAD INSPECTION AND DISTRESS MEASUREMENT PROCEDURES

The unsurfaced road network must be divided into branches, sections, and sample units before it is inspected and distresses are measured. If a pavement management system for paved roads is being used, the same procedures can be followed for unsurfaced roads. If the PAVER PMS is being used, the specifications in Army Technical Manual 5-623, mentioned earlier, should be followed. Once this has been accomplished, road condition data can be obtained and the URCI of each section can be determined.

A windshield inspection and detailed distress measurements are both performed for unsurfaced roads. A description of the recommended inspection and distress measurement procedures follows.

Unsurfaced road inspections should be made from inside the road agent's vehicle at 25 mph. The inspector should drive the full length of each unsurfaced road, and note any surface distresses and drainage problems. These inspections should be made four times a year, once during each season. However, detailed distress measurements necessary to compute the URCI are not required every year. These field measurements should be taken between 15 August and 15 September in order to compare ratings from one year to the next. This time period is based on conditions in New England and may vary for other parts of the United States. It is the time of the year in which roads in New England are in the best and most consistent condition from year to year.

Measurement sample units should be 100 ft long, and the number of samples measured per section depends on the length of the section. Two sample units per mile of unsurfaced road generally are sufficient.

Data collected during the distress measurements are used to calculate the URCI, which is based on deduct-values. As was previously stated, a deduct-value is a number from 0 to 100, in which 0 indicates that the distress has no effect on the road condition and 100 indicates that the road has completely failed.

The URCI of a sample unit can be calculated by the following simple, five-step procedure:

1. Each sample unit selected for distress measurements is inspected and distress data are recorded on the Unsurfaced Road Inspection Sheet (Figure 7).
2. The deduct-values are determined from the deduct-value curves for each distress type and severity level.
3. A total deduct-value (TDV) is computed by summing all individual deduct-values.

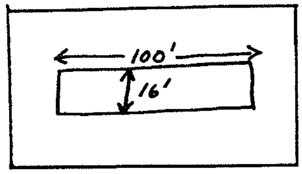
SARDIS LAKE, CLAYTON, OKLAHOMA

Branch	POTATOE HILL CENTRAL	Section	1
Date	07/15/86	Sample Unit	1
Surveyed By	R. EATON .	Area of Sample	100' x 16'

DISTRESS TYPE

1. Improper Cross-Section (linear feet)
2. Roadside Drainage (linear feet)
3. Corrugations (square feet)
4. Dust (table)
5. Potholes (number)
6. Rutting (square feet)
7. Loose Aggregate (linear feet)

SKETCH



EXISTING DISTRESS TYPE QUANTITY & SEVERITY

TYPE		1	2	4	5	7					
QUANTITY & SEVERITY		100LF	100LF	LOW	1 MED	25LF					
		LOW	HIGH		1 LOW	HIGH					
			100LF			100LF					
			MED			LOW					
TOTAL	L	100		✓	1	100					
	M		100		1						
SEVERITY	H		100			25					

URCI CALCULATION

DISTRESS TYPE	DENSITY	SEVERITY	DEDUCT VALUE
1	6.3	L	13
2	6.3	M	15
2	6.3	H	20
4		L	2
5	0.1	L	1
5	0.1	M	4
7	6.3	L	10
7	1.6	H	8
q = 5	TOTAL DEDUCT VALUE		73
CORRECTED DEDUCT VALUE (CDV)			36

URCI = 100 - CDV = 100 - 36 = 64

RATING = GOOD

REMARKS:

FIGURE 7 Unsurfaced road inspection.

4. Once the TDV is computed, the corrected deduct-value (CDV) can be determined from a correction curve (Figure 8). If any individual deduct-value is higher than the CDV, the CDV is set equal to the highest individual deduct-value.
5. The URCI is computed using the relation $URCI = 100 - CDV$.

The URCI for a section is computed by taking the arithmetic mean of all the individual URCIs of all sample units measured.

EXAMPLE OF URCI CALCULATION

A sample section called Potatoe Hill Central at Sardis Lake, Clayton, Oklahoma, was chosen to illustrate the determination of the URCI (Figure 7).

Based on the previously described procedures, the URCI of Potatoe Hill Central is calculated as follows.

Step 1. Each sample unit has been inspected and the distress recorded on an Unsurfaced Road Inspection Sheet (Figure 7).

Step 2. The deduct-values are determined from the deduct-value curves. The densities of each distress and severity level are based on a sample unit of 1,600 ft^2 (shown in Figure 7).

1. For 100 linear ft of a low-severity improper cross-section, the density equals

$$\frac{100}{1,600} \times 100 = 6.3.$$

2. For 100 linear ft of medium-severity roadside drainage, the density equals

$$\frac{100}{1,600} \times 100 = 6.3.$$

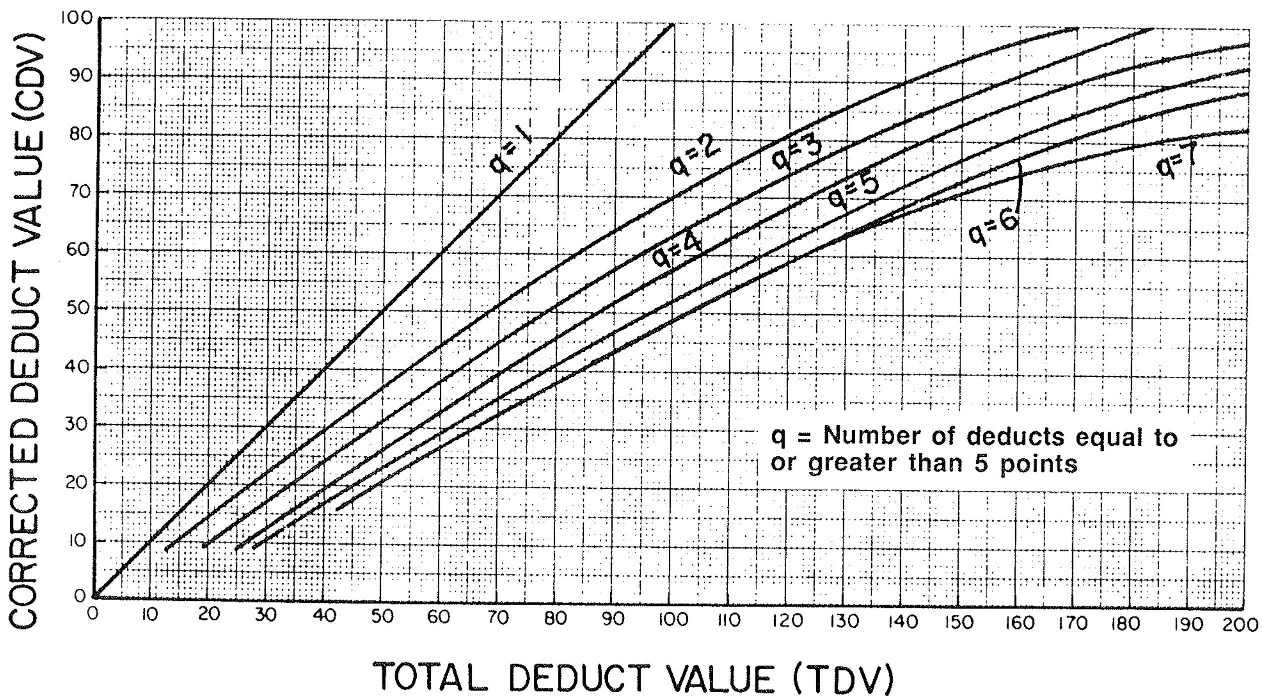


FIGURE 8 Correction curves.

3. For 100 linear ft of high-severity roadside drainage, the density equals

$$\frac{100}{1{,}600} \times 100 = 6.3.$$

4. Dust has been measured as low severity.

5. For one pothole at low severity, the density equals

$$\frac{1}{1{,}600} \times 100 = 0.1.$$

6. For one pothole at medium severity, the density equals

$$\frac{1}{1{,}600} \times 100 = 0.1.$$

7. For 100 linear ft of low-severity loose aggregate, the density equals

$$\frac{100}{1{,}600} \times 100 = 6.3.$$

8. For 25 linear ft of high-severity loose aggregate, the density equals

$$\frac{25}{1{,}600} \times 100 = 1.1.$$

Using the deduct-value curves, deduct-values can be obtained for all the densities computed above; these are shown in Figure 7.

Step 3. A TDV is computed for the sample unit. For example, in Step 2 the total deduct-value is 13 + 15 + 20 + 2 + 1 + 4 + 10 + 8 = 73.

Step 4. The CDV is computed. In the example in Step 2, the TDV was found to be 73. The value of q, the number of individual deducts the value of which is 5 or greater, is 5. Based on the corrected deduct-value curve in Figure 8, the CDV is 36.

Step 5. The sample unit URCI is computed using the relation $URCI = 100 - CDV$. In this example the URCI = 100 - 36 = 64; the rating is good. Note that if the section being rated had had only one sample unit, the section's URCI would also have been 64. But if two or more sample units had been rated, the section's URCI would have been the arithmetic mean of all of the sample units rated.

MAINTENANCE MANAGEMENT

The ratings obtained using this procedure can be used to effectively manage maintenance of unsurfaced roads. Each agency can establish critical URCI ratings that can be used to establish a maintenance strategy. For example, a rating of 50 on a road would require the development of maintenance action to restore the road to a rating of 75 or higher. This technique could be used as a stand-alone, or manual, pavement management system, or it could be used in conjunction with PAVER or any other automated PMS. The integration of this rating method into PAVER would provide procedures to divide the road into sections, conduct a road condition survey and rating, evaluate a road, determine rational maintenance and repair needs and priorities, perform life-cycle costing on feasible maintenance and repair alternatives, and develop manual or automated systems to store and retrieve data.

CONCLUSIONS

A method of rating unsurfaced roads has been developed and field-validated at seven test areas across the United States from New England to Alaska. This method can be used alone to rate unsurfaced roads, or it can be incorporated into automatic, computer-aided pavement maintenance management systems for paved roads, such as PAVER.

Manual or computer-aided PMS use of this rating method should provide the data necessary for optimum allocation of resources and maintenance of unsurfaced roads in the best possible condition for the least cost.

ACKNOWLEDGMENTS

The authors would like to thank all workshop participants, evaluation panel members, steering committee members, and sponsoring agencies noted in this paper for their tremendous support of this project. Special thanks are due to S. Alford, R. Beaucham, W. Benton, E. Blackmon, J. Glenn, R. Hauger, T. Houston, N. J. LeBrun, K. Leland, F. Oler, R. Stith, and W. Wilkerson for their participation in the field validation phase of this project.

Pavement Management for Low-Volume Roads

CAROL W. BLAIR, EDWARD G. BATES, JR., AND DAVID M. DREVINSKY

Pavement Management for Communities is a manual for small road networks. Every road agency with maintenance responsibilities is experiencing the problem of escalating costs and deteriorating road conditions; pavement management is a solution. However, many smaller communities do not have the resources to implement the pavement management methods offered in an abundance of literature on the subject. Some methods require extensive data. Others require the use of a computer. Most involve a significant amount of time to understand the methodology and collect data, or a considerable investment in outside services. The goals of the manual discussed in this paper are to introduce local officials and highway superintendents to the concept and benefits of pavement management, and to distill the extensive work of others into a simplified approach to pavement management. The alternatives begin with a basic, stripped-down method, suitable for situations that demand a quick turn-around with a minimum of resources. The basic method is presented in detail and appropriate charts and forms are included. Possible refinements are then discussed and modifications are offered to include additional factors or to gain precision. Available information on pavement management software and consultants is included. Communities are encouraged to adapt these methods to best suit their particular needs and resources.

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This paper is based on the premise that pavement management is important for low-volume roads. Although some jurisdictions may have adequate maintenance budgets, others regularly defer part of their maintenance program because of inadequate funding. The costs of deferring maintenance are significant and should be addressed in the process of budgeting for maintenance activities. Furthermore, ranking road maintenance projects systematically, with the goal of minimizing long-term maintenance expenditures, is essential in cases in which a budget shortfall exists. A description is provided of work performed by the Metropolitan Area Planning Council (MAPC) (Boston) in response to a need among member communities to formalize the pavement management process.

After it was recognized that limited resources were available for such an activity, a simplified manual was developed to demonstrate how to document maintenance needs and program needed improvements. The manual is based on the synthesis of existing pavement management manuals and the seasoned advice of a panel whose members were drawn from universities, consulting firms, and government. There are no new or global solutions to the problems of inadequate budgets and deferred maintenance, but the simple methods described offer the tools needed to justify increased funding and to effectively spend the funds that are available.

THE NEED FOR PAVEMENT MANAGEMENT

Pavement management is the process of overseeing the maintenance and repair of a network of roadways. Unfortunately, pavement management programs and their required funds are generally not adequately documented. This makes road maintenance funding proposals especially vulnerable to budget cuts, and even meager funding requests are often deferred.

The costs of deferring maintenance are great. Poorer road conditions result in higher vehicle maintenance costs, reduced safety, and loss of rideability. Furthermore, a deferred project is likely to cost more later because of inflation. By the time it is implemented, the proposed project may be inadequate to rehabilitate the further deteriorated road.

The latter point is critical to pavement management, and is illustrated by Figures 1 and 2 (*1*). Note that the cost of renovating a road at 75 percent of its service life may be as little as 20 percent of the cost of the renovation deferred to the point at which the road has reached 87 percent of its service life. Timely maintenance is obviously fundamental to effective pavement management.

The literature on pavement management and the software developed to date have been excellent. Both have provided well-reasoned methods to survey, analyze, and program any system of roads or pavement. However, the resources required to implement some of these methods are beyond what many jurisdictions are prepared to invest, at least until the value of pavement management is proven to local officials.

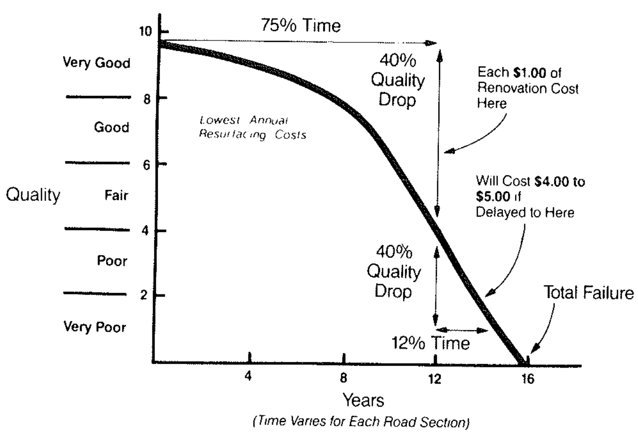


FIGURE 1 The cost of timely maintenance.

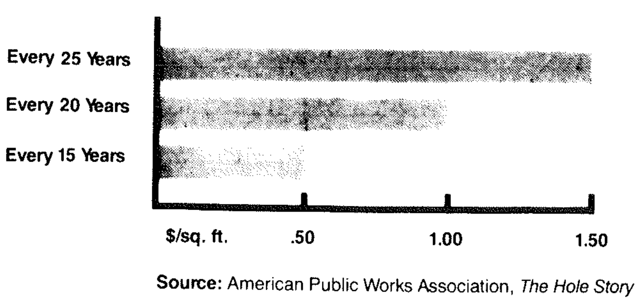


FIGURE 2 Annualized cost to overlay every 15, 20, and 25 years.

PAVEMENT MANAGEMENT FOR LOW-VOLUME ROADS

Pavement management has many applications, and each deserves a different response. An agency that has a significant backlog of maintenance work, many roadways in poor condition, and little or no experience in pavement management needs a simple method to summarize maintenance needs and document priorities. A jurisdiction that has an effective program of pavement maintenance can use pavement management to make more cost-effective decisions at the project level. This situation requires more detailed data and more sophisticated methods. It is likely that, for many jurisdictions, the pavement management process will evolve from the first effort to harness a runaway problem of escalating costs and deteriorating roads to a more sophisticated position of optimizing maintenance costs and road conditions.

The 101 member communities of the MAPC include both urban and rural communities with large and small road systems; most of these communities are facing the problem of reduced budgets and deteriorated roads. The MAPC offered *Pavement Management: A Manual for Communities* to these communities to present diverse pavement management options. The goal of the manual is to provide a basic method that any road maintenance organization would be able to use, and a complete selection of options for refinements.

The manual was developed with extensive participation by experts and potential users. A technical advisory committee of 10 active members involved in research, consulting, and highway administration provided valuable information and insights. Three communities participated in testing the procedures of the manual, and about 50 communities participated in the training workshops that followed.

The manual is organized in five chapters. After the introduction, the second chapter asks the reader "What Can a Pavement Management Program Do for You?" and includes AWPA's *The Hole Story* with concise and dramatic arguments for maintenance programming. Chapter 3, "Pavement Management Made Simple," provides a basic method for dealing with road maintenance needs. Chapter 4, "Refinements," offers alternative techniques for greater precision in each of the steps involved in programming. The pavement management experiences of five communities are reported in the last chapter. The remainder of this discussion centers on the methods offered in this manual.

PAVEMENT MANAGEMENT MADE SIMPLE

A basic method of developing a pavement maintenance program is presented in Chapter 3 of the manual. This method can be used by superintendents who cannot devote a lot of time to planning, but who recognize that maintenance needs must now be documented in order to procure adequate funds.

The five steps presented in this chapter are flexible and can be tailored to individual needs. This method could easily be computerized using commonly available spreadsheet software.

The five steps suggested in this chapter are shown in Figure 3. Step 1 is the production of a street inventory that defines the street network by segments. Step 2 is the survey of pavement conditions and the documentation of required maintenance for each street segment. Step 3 is the ranking of projects to ensure that the most severe problems and the most cost-effective projects are considered first. Step 4 involves the scheduling and funding of the work to be performed. Step 5 is the implementation of the program; it represents the feedback between maintenance needs and fiscal resources. This step also relates the program to the realized outcome (work completed). Good record-keeping practices are an essential component of this process.

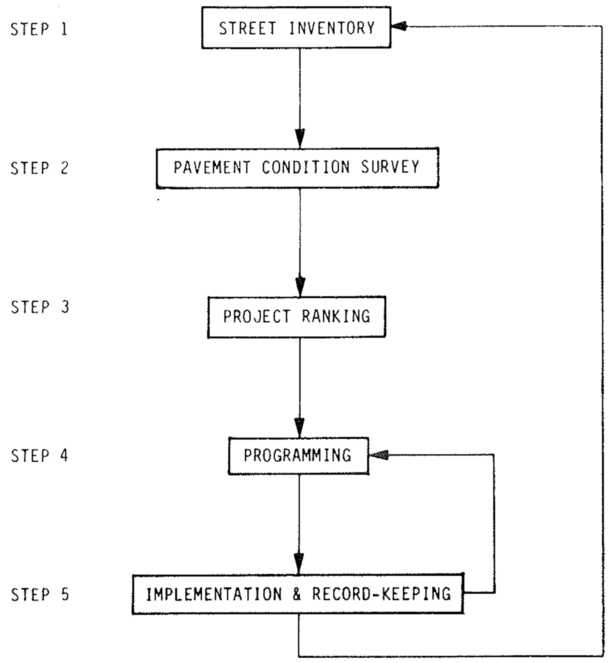


FIGURE 3 Pavement management in five steps.

Step 1: Street Network Inventory

The inventory is a list of street names and their corresponding length and width. A sample data form is shown in Figure 4. Surface type (i.e., paved or unpaved) should be included in the initial survey. In addition, a system for dividing the road network into manageable segments must be devised. A simple approach is to designate sections that correspond to intersections or to changes in pavement condition. Sections can be identified by house number, street name, or any other device, provided the landmark is permanent.

Step 2: Pavement Condition Survey

The pavement condition survey should collect the information needed to identify the following:

- Streets that need no immediate maintenance and therefore no immediate expenditures;
- Streets that require minor or routine maintenance and immediate expenditures;
- Streets that require preventive maintenance activities such as asphalt overlays or seals; and
- Streets that need major rehabilitation or reconstruction. These roads have deteriorated to the point that maintenance is no longer cost-effective and more major work is required to raise the condition to an acceptable level (*2*).

The sample condition survey form shown in Figure 5 is a simple tool for gathering the survey data. This form assumes the same section numbers that were noted on the previous street inventory form (Figure 4). Pavement condition is identified from one of the six levels described on the form, so that the inspector can refer to the definitions if, for example, there is doubt as to whether the pavement is in fair or poor condition. Drainage is rated from 1 to 3 in the same fashion, using qualitatively defined conditions.

The inspector should take advantage of the space provided for comments to record any observations that might affect the work to be recommended. For instance, if the pavement is rated at condition C and appears to have deteriorated faster than was expected because of a drainage problem, this should be noted. In this case a plan for treating the drainage problem would be a necessary part of maintaining the roadway.

The recommended action is an essential part of the condition survey and can be inferred from the graph shown on the survey form. If the inspector has considerable experience in pavement maintenance, the recommendation may reflect relevant factors not specified in this form, such as obvious safety hazards or a poor road base. These other factors should also be noted.

Inspector's Name ______________________ Date ____________

Street ____
(name and section number)

From: ____________________

To: ____________________

Length: ________ Average Width: __________

Traffic (circle one)

Low Medium High

Trucks (circle one)

Low Medium High

Surface Type (circle one)

Aggregate Bituminous Pavement Concrete Pavement

FIGURE 4 Sample pavement inventory form.

Street Name & Section Number: ____________________

Inspector's Name: ____________________ Date: ______

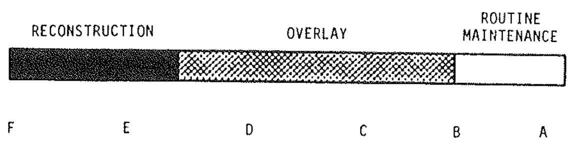
Pavement Condition: (circle one)

A.	Excellent	Little distress. New or nearly new pavement.
B.	Good	Significant distress. Treatable with sealing and patching.
C.	Fair	Moderate distress. Deteriorating rapidly.
D.	Poor	Extensive distress. Thin overlay may be ineffective.
E.	Very Poor	Near failure.
F.	Failure	Dangerous. Requires constant repair.

Drainage Conditions:

1. Good: Ditches, culverts, inlets clean. Road shoulders slope down away from roadway in most places.
2. Fair: Ditches, culverts, inlets fairly clean. Road shoulders slope down away from roadway in most places.
3. Poor: Ditches not clean, culverts and inlets clogged. Road shoulders are often higher than the roadway.

Recommended Action (circle one)



Year when this work should take place:

COMMENTS:

FIGURE 5 Sample pavement condition survey form.

The year specified for the proposed maintenance or improvements is also important. The inspector should estimate the best time to perform the work and, if possible, include a comment about the alternatives. For instance, the recommendation might be to resurface (overlay) in year 2, with the comment that if the overlay is not in place within 3 years, reconstruction of the pavement and base will be required.

Step 3: Project Ranking

When the pavement survey is complete and maintenance needs have been determined, the next step is to rank the recommended maintenance actions for specific street segments. The philosophy of project ranking reflects both the worst-first and best-first concepts. The pavements in the poorest condition clearly have high priority. These sections cause unnecessary wear and tear to vehicles, are expensive to maintain, and may be hazardous. However, the best roads, those that are well-built and in good condition, represent an investment that should be protected against normal deterioration.

To satisfy the need to set dual priorities, the worst-first criterion is applied within each type of maintenance: rehabilitation and reconstruction. No priorities are set for routine maintenance, which is presumably accomplished within adequate force accounts. The best-first criterion is then used in the programming stage (Step 4) to ensure that routine and preventive maintenance is not short-changed in favor of the more conspicuous reconstruction projects. A separate list of ranked projects should be developed for rehabilitation and reconstruction. The trade-offs between these two categories are a matter of policy, set in programming (Step 4). Again, routine maintenance should not be a priority and should be funded as a group before any other projects.

If one regards pavement condition as the sole criterion for ranking projects, it is not necessary to use the following scoring formula and one should refer to Step 5. If projects are ranked according to traffic loads, a priority score can be estimated from survey information on pavement condition, traffic volume, and truck traffic. The formula for the priority score, P, is as follows:

$$P = PC \times (TV + TT) \quad (1)$$

where

PC = pavement condition,
TV = traffic volume, and
TT = truck traffic.

Note that this formula requires that descriptive information from the survey be translated into numeric values, as shown in Table 1. An example of the priority list for rehabilitation projects is shown in Table 2.

The three lists of projects that result for routine maintenance, rehabilitation, and reconstruction form the basis for the rational programming of funds in Step 4.

TABLE 1 NUMERIC CODES FOR SURVEY INFORMATION

Descriptive Information	Survey Description	Numeric Value
Pavement Condition	A Excellent	1
	B Good	2
	C Fair	3
	D Poor	4
	E Very poor	5
	F Failure	6
Traffic Volume	Low	1
	Medium	2
	High	3
Truck Traffic	Low	1
	Medium	2
	High	3

Step 4: Programming

After all maintenance needs and their relative priorities are listed within each type of maintenance project, a decision must be made on where to spend the limited funds available and whether additional funds should be appropriated.

The cost of each project must first be estimated. In this planning stage, approximate unit costs are sufficient. A list of unit costs that were developed for one pavement management program is shown in Table 3. Other examples are provided in the appendix of the manual.

Each jurisdiction should make a short list of unit costs for treatments that were used recently. This will avoid confusion concerning the procedure being estimated, changing costs over time, and local price differences. It may be convenient to specify average unit costs per mile for specific procedures, such as crack

TABLE 2 A RANKED LIST OF REHABILITATION PROJECTS

Year	Street	Pavement Condition	Traffic Volume	Truck Traffic	Priority Score (P)
1	Main	4	3	2	20
1	Maple	4	2	2	16
1	Washington	3	2	3	15
2	School	4	2	1	12
2	Cross	3	1	2	9
2	Hill	3	1	1	6
3	Woodridge	2	2	1	6
3	Holly	2	1	1	4

TABLE 3 SAMPLE UNIT COSTS FOR PROPOSED MAINTENANCE (*3*)

Treatment Type	Description	Cost per Square Yard ($)
Reconstruct	Full depth, local street	9.08
Reconstruct	Full depth, collector street	12.49
Reconstruct	Full depth, arterial street	16.98
Reconstruct	Pavement reclamation	8.28
Rehabilitation	Leveling course and overlay	4.98
Rehabilitation	1-1/2 in overlay	2.57
Rehabilitation	5 percent patch and crack seal, then chip seal	3.23
Rehabilitation	20 percent patch and overlay	5.21
Rehabilitation	Cold planing and overlay	5.57
Rehabilitation	Crack seal and overlay	2.98
Maintenance	5 percent patch and crack seal, then chip seal	1.93
Maintenance	Chip seal with crack seal	1.27
Maintenance	Crack seal low	0.41
Maintenance	Crack seal high	1.26
Maintenance	5 percent patch	0.66
Maintenance	20 percent patch	0.64
Maintenance	Patch and seal	0.54

sealing, 1-1/2 in overlay, and reconstruction to 12 in. These costs can then be easily applied to the road segments measured in Step 1 to yield rough estimates of the project costs.

Project costs can be summed within each maintenance category to estimate total dollar needs. Comparison of these dollar needs with currently available funds will raise the necessary programming questions of whether additional funds can be allocated to the program and over how many years this program can be spread. Even if the first question is never finally answered, a clear maintenance program can provide the information needed for budget decisions and for lobbying to increase funding for road maintenance.

The second question is more technical and must be answered by the road superintendent. The finance committee can benefit from a prospective look at the long-term future, which could be about 10 years. However, projections for maintenance after about 5 years may be of value only at the network level. Answering these questions is an iterative process.

The first round continues with the assignment of funds to each category. For instance, the initial policy may be to fund 100 percent of routine maintenance, 80 percent of rehabilitation work, and 40 percent of reconstruction projects in the first year. The result is that 20 percent of the rehabilitation work and 60 percent of reconstruction work must be postponed to the second year.

As projects are assigned to the work program for the first year, the second year, and so on, the penalties for postponing work are felt. For each of the deferred projects, routine maintenance must be funded for the current year. Furthermore, the original recommendation may require revision. If, for instance, a street recommended for an overlay in year 1 is deferred to year 4, it will most likely require reconstruction by the time the work is to be performed.

If the resulting program promises to maintain the street network at the current level of service, then the program is complete. Once funded, it is ready for implementation (Step 5).

If the resulting program indicates that the condition of roads and the level of service provided will decline, on average, over the course of the program, then the programming process has been invaluable. Without it, the current levels of maintenance funding and projects would have led to a system of failing roadways. That could be disastrous in economic terms because it could take four or five times greater expenditures to rebuild after a failure than it would have taken to rehabilitate only a few years earlier. This program obviously should be presented to the mayor together with a second program proposing an increase in street maintenance funds to maintain the system properly in the coming years.

Step 5: Implementation and Record-Keeping

The feedback process is important in pavement management. The first list of maintenance needs developed by the superintendent must respond to fiscal limitations. Repeated adjustments are then required to achieve the program that will buy the most in terms of long-term pavement service with the resources the local government has allocated.

The approved program can then be implemented. Further adjustments may be necessary as a result of delays in contract work, unforeseen maintenance problems, and so forth. In any case, the program should be updated every year or two to reflect both work completed and further deterioration of pavements.

An essential part of the updating process is good record-keeping. A street-by-street file for tracking pavement condition and maintenance actions over the years should include a record for each street segment (see Figure 6). Maintenance should be recorded as it is performed. Data on pavement conditions can be updated as staff time is available. Once the programming process is established, updating the data and recommendations can be a routine function.

REFINEMENTS

The basic method presented in Chapter 3 can be refined in many ways, including incrementally, if necessary. The refinements offered in Chapter 4 include expanded inventory, details of pavement condition, drainage problems, economic analysis, maintenance alternatives, and computerization.

Expanded Inventory

A simple form is offered in Chapter 3 to record the following:

- Street name and segment,
- End points,
- Length and width,
- Total traffic and truck traffic, and
- Surface type.

Any of several other attributes of the road could be relevant in setting maintenance priorities, including drainage, traffic capacity, and safety factors. A sample form is presented in Figure 7 that can be used in a comprehensive inventory.

This added information can be used to rank projects, as demonstrated in Chapter 3. Each measure is translated into a score that indicates adequacy and is then used as a multiplier in the priority score. For example, the following formula for priority, P, is given in Chapter 3:

$$P = PC \times (TV + TT) \tag{1}$$

where

PC = pavement condition,
TV = traffic volume, and
TT = truck traffic.

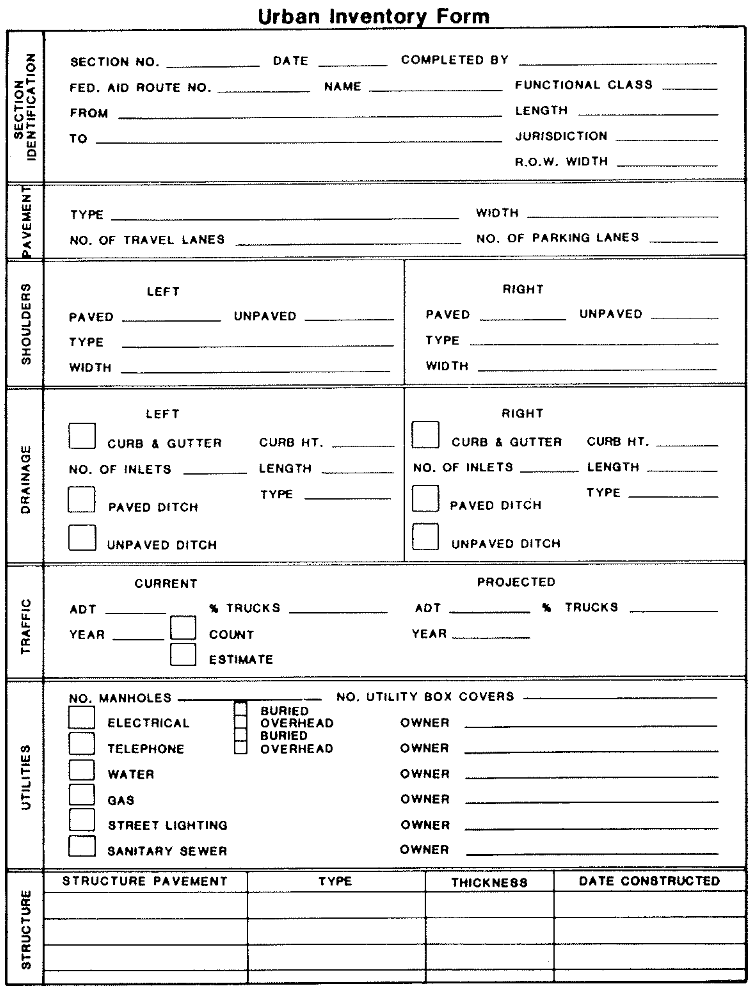
Maintenance Summary Form

SECTION NUMBER ____________ YEAR CONSTRUCTED ____________

MAINTENANCE ACTIVITY	PAVEMENT TYPE	RATE OR THICKNESS	DATE	UNIT COST

MAINTENANCE ACTIVITY - OVERLAY CHIP SEAL
REHABILITATION
RECONSTRUCTION
OTHER (DESCRIBE)

FIGURE 6 Sample maintenance summary form (*2*).



Urban Inventory Form

SECTION IDENTIFICATION
SECTION NO. ____ DATE ____ COMPLETED BY ____
FED. AID ROUTE NO. ____ NAME ____ FUNCTIONAL CLASS ____
FROM ____ LENGTH ____
TO ____ JURISDICTION ____
R.O.W. WIDTH ____

PAVEMENT
TYPE ____ WIDTH ____
NO. OF TRAVEL LANES ____ NO. OF PARKING LANES ____

SHOULDERS
LEFT: PAVED ____ UNPAVED ____ TYPE ____ WIDTH ____
RIGHT: PAVED ____ UNPAVED ____ TYPE ____ WIDTH ____

DRAINAGE
LEFT: CURB & GUTTER, CURB HT. ____, NO. OF INLETS ____, LENGTH ____, TYPE ____, PAVED DITCH, UNPAVED DITCH
RIGHT: CURB & GUTTER, CURB HT. ____, NO. OF INLETS ____, LENGTH ____, TYPE ____, PAVED DITCH, UNPAVED DITCH

TRAFFIC
CURRENT: ADT ____ % TRUCKS ____ YEAR ____ COUNT ESTIMATE
PROJECTED: ADT ____ % TRUCKS ____ YEAR ____

UTILITIES
NO. MANHOLES ____ NO. UTILITY BOX COVERS ____
ELECTRICAL (BURIED / OVERHEAD) OWNER ____
TELEPHONE (BURIED / OVERHEAD) OWNER ____
WATER OWNER ____
GAS OWNER ____
STREET LIGHTING OWNER ____
SANITARY SEWER OWNER ____

STRUCTURE

STRUCTURE PAVEMENT	TYPE	THICKNESS	DATE CONSTRUCTED

FIGURE 7 Sample comprehensive pavement inventory form (2).

Drainage can also be given a score of 1 to 3; the formula would then be as follows:

$$P = PC \times (TV + TT) \times D \qquad (2)$$

where

D = an index of drainage.

Any relevant measure of adequacy can be included in the ranking scheme in this manner.

Other items on the expanded inventory can be used for other purposes. Records of utilities and structures, for instance, are helpful in determining the most appropriate maintenance alternative and cost, and the remaining life of the pavement.

Great care should be taken to include any information that could be important to the program, and, at the same time, to avoid the collection of data that will not be utilized.

Details of Pavement Condition

In many cases it is desirable to go beyond the simple A to F classification of pavement conditions in order to ensure rating consistency. This is especially true if several individuals will be rating the pavement. A more objective measure can be achieved by rating the pavement quantitatively on each of several aspects of pavement condition. A sample of commonly surveyed distress types follows:

- Alligator Cracking — a series of interconnecting cracks resembling alligator skin or chicken wire.
- Bleeding — a film of bituminous material on the pavement surface which creates a shiny or glasslike appearance.
- Block Cracking — cracks which divide the surface into approximately rectangular pieces.
- Corrugation — ripples across the asphalt surface resulting from plastic movement.
- Joint Reflection Cracking — cracks in asphalt concrete which coincide with joints of underlying PCC slabs.
- Longitudinal Cracking — cracks which are parallel to the pavement centerline or laydown direction.
- Polished Aggregate — aggregate which has lost its rough irregular texture.
- Pothole — a bowl-shaped hole in the pavement surface.
- Pumping — ejection of water and fine materials through cracks under pressure of moving loads.
- Rutting — a surface depression in the wheel paths.
- Slippage Cracking — crescent or half-moon shaped cracks resulting from sliding or deformation of the pavement.
- Swell — an upward bulge in the pavement.
- Transverse Cracking — cracks perpendicular to pavement centerline.

Several excellent catalogues of pavement distress are also available, and are identified in the appendix of the manual. These references provide both descriptions and photographs of each type of pavement distress or failure. In some cases, causes and repair techniques are also addressed. The literature includes three authoritative methods for condition surveys, each of which is described briefly here, and more extensively in the manual.

The Asphalt Institute method includes a condition rating that ranges from 0 to 100 (*4*). Thirteen different types of distress are evaluated, each on a scale of 0 to 5 or 0 to 10, and then subtracted from 100 to yield the condition rating. This technique is easy to use, but is somewhat subjective. It also assumes that each type of distress should be weighted equally in every situation. For example, shoving and pushing are always responsible for 10 percent of the overall condition rating.

The Federal Highway Administration method assesses eight different forms of distress and overall riding quality (*2*). Visual estimates are required for each distress type to characterize the severity of the distress as slight, moderate, or severe and the extent of the distress as a percentage of roadway area. This approach is far more objective than others, but it also requires more survey time. Furthermore, the scoring key to translate this data into a distress condition rating is not provided but must be developed by the user.

The Army Corps of Engineers method uses physical measurements of 19 types of distress at low, medium, and high levels of severity (*5*). This method entails the greatest amount of data collection and is the most precise method of the three described here. It has been adopted and computerized by the American Public Works Association and is offered to member communities on a time-sharing basis at cost.

A comparison of the three methods just described is shown in Figure 8. One of the attributes shown is roughness, or rideability. Rideability is a measure of riding comfort and is measured subjectively on a scale from 0 to 5. Roughness is a corresponding mechanical measure that is made by a wheel suspension device. Other mechanical devices can also be used to make measurements more precisely, such as the Benkelman Beam and deflection meters.

Automation of the road survey is also possible through the use of a computerized van with optical or laser scanning capabilities. References to literature and consultants are provided in the manual.

Most communities will begin with visual surveys of distress, and possibly take advantage of the methods provided by the Asphalt Institute, Federal Highway Administration, or American Public Works Association. Mechanical and automated methods may be more appropriate to larger networks, in which the expense of such techniques is spread over more miles and the advantages of standardization are greatest.

Many other methods are available through the literature, engineering consultants, and computer software vendors. A review of these resources is recommended before beginning an elaborate pavement management system.

Maintenance Alternatives

The most appropriate maintenance and the timing of the work are both critical in maximizing cost-effectiveness. The highway superintendent is undoubtedly aware that many pavement treatment options are available. The manual includes a description of alternative seals and mixes, and comments on their performance and service life. Although it is practical to work with just a few of these in any one community, a continued effort should be made to evaluate their performance and the potential of alternative treatments.

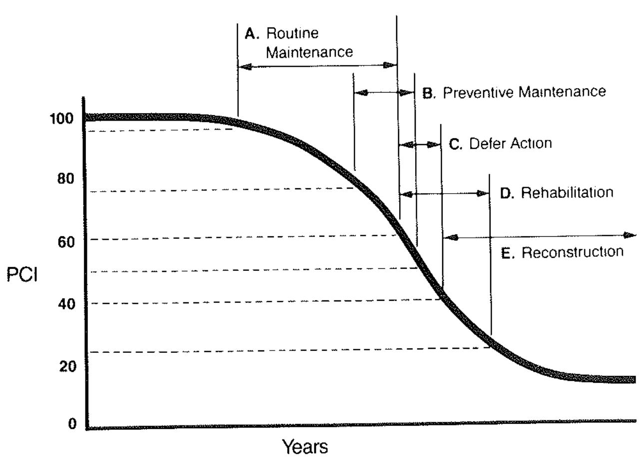
Timing of maintenance treatments is critical, as shown in Figure 9. Note that routine and preventive maintenance are appropriate on the most comfortable part of the curve, in which the pavement is still in good condition. When the pavement begins to deteriorate more quickly, rehabilitation or even reconstruction is usually required.

In order to develop systematic assignments of treatment, cost, and value for elements of the street network, it is convenient to describe the menu of maintenance treatments in the five categories shown in Figure 9 and described below. The following is an excerpt from *Road Surface Management for Local Governments* (*2*).

> *Routine Maintenance*—For roads in reasonably good condition, routine maintenance is generally the most cost-effective use of funds. If at all possible, all routine maintenance needs should be funded each year. Routine maintenance usually includes local patching, crack sealing, and other relatively low-cost actions. Distresses such as isolated medium or high severity bumps or potholes that may have a considerable negative impact on the performance of a section are usually corrected first.
>
> *Preventive Maintenance*—This strategy is a more expensive activity designed to arrest deterioration before it becomes a serious problem. Surface seals are excellent examples of preventive maintenance. A common source of poor performance of seals is inadequate repair of existing distress before sealing, so extensive repair work may also be included in preventive maintenance. Repair and seal needs will probably have to be programmed over several years in order of priority because of the expense. Routine maintenance should be performed on those sections that are not programmed for the current budget year.
>
> *Deferred Action*—The road sections which fall into this category receive minimum funds for the current budget year. These sections are beyond the point where preventive maintenance will be effective but have not yet deteriorated to the point of needing rehabilitation. Selecting this strategy is deferring action, so an agency must be prepared to fund rehabilitation or reconstruction when it becomes necessary. This strategy is normally not appropriate for aggregate surfaced roads.
>
> *Rehabilitation*—Rehabilitation usually includes overlays or extensive recycling. Funding for completion of these major projects may depend upon federal or other outside sources. The established priorities should be followed if possible, although managers should realize that priorities may change for a variety of reasons. For example, estimates for a particular job may exceed available funds, or insurmountable administrative restrictions on funds may exist, or very valid political reasons to change priorities may occur. Sections that fall into this strategy category that are not programmed for the current budget year should fall into the deferred action strategy.
>
> *Reconstruction*—The comments on rehabilitation projects also apply to reconstruction projects. The main difference is in the costs that might be expected. Reconstruction would involve complete removal and replacement of a failed pavement and might also involve features other than just pavement, such as widening, realignment, traffic control devices, safety hardware, and major drainage work. Lead times of five to ten years might be required because of the significant nature of required investments and the time necessary to develop plans, acquire right-of-way, and other funding.
>
> There is considerable overlap of possible strategies on the performance curve. In the example shown, there are two or three possible strategies for any point in the mid-range of pavement conditions. This is a very realistic approach because the deterioration of pavements is a gradual process. A small change will not usually make one strategy preferable over another.
>
> The following priority groups should constitute the program developed from these five treatment strategies:

Technique	Asphalt Institute	Federal Highway Administration	Army Corps of Engineers
Pavement Characteristics			
• Number observed	13	9	19
• Cracking	transverse longitudinal alligator shrinkage	transverse longitudinal alligator	longitudinal & transverse alligator block edge joint reflection slippage
• Shifting	rutting corrugation shoving or pushing	rutting corrugation	rutting corrugation shoving depression swell bumps & sags
• Separation	raveling excess asphalt polished aggregate pot holes	raveling flushing	raveling & weathering bleeding polished aggregate pot holes lane/shoulder drop-off
• Patching		patching	patching & utility cut
• Roughness	overall riding quality	riding quality	
• Drainage	deficient drainage		
• Other			railroad crossing
Method of Rating Severity of Distress	Scales of 0 to 5 or 0 to 10	slight, moderate, severe	low, medium high
Method of Rating Extent of Distress	Reflected in estimate of severity	1-15%, 16-30% 31%+ (of area)	square feet linear feet # of pot holes
Combining Measures of Distress to Yield an Index of Pavement Condition	Simple addition gives index with range from 0 to 100	Sample scoring Key is provided (Not necessarily appropriate to the system being evaluated)	Scoring is complex and more appropriate to mainframe computer than hand calculations
Adopting the Rating System	Distress items can be added or deleted, and scales expanded for emphasis	Scoring is complex and adaption requires experience with the model	Scoring is complex and adaption requires experience with the model

FIGURE 8 Comparison of pavement condition rating techniques.



Source: *Road Surface Management for Local Communities Course Workbook.* U.S. Department of Transportation and Federal Highway Administration, May, 1985.

FIGURE 9 Maintenance strategies and timing (*2*).

A. Routine Maintenance (It is probably not worthwhile to determine priorities but rather just list sections in this strategy.)
B. Preventive Maintenance
   Priority Group 1
   Priority Group 2
   Priority Group 3
C. Deferred Action: No priorities are necessary, just a list of sections.
D. Rehabilitation
   Priority Group 1
   Priority Group 2
   Priority Group 3
E. Reconstruction
   Priority Group 1
   Priority Group 2
   Priority Group 3.

Economic Analysis

Economic analysis is a powerful technique for the objective evaluation of alternatives. It can be used in pavement management to establish the most cost-effective maintenance treatments and to compare the relative priority, or value, of alternative projects. The measurement of a variety of factors by a common unit, the dollar, makes possible an objective, quantitative comparison of treatments, projects, and schedule alternatives. Unfortunately, these comparisons require a number of assumptions about the future value of today's dollar, the expected life of capital, and the value of time.

A summary of the methods often used in economic analysis is included in the appendix of the manual. These techniques are especially appropriate to large systems, in which the savings from a complete analysis make the added complexity worthwhile.

Computerization

If the superintendent has, or is planning to have, access to a personal computer, it is worthwhile to think about using it for pavement management. The programming process involves repetitious sorting and arithmetic. Once the process is set up, the computer can make revisions and updates a simple matter.

Two options are available. The first is to use commonly available spreadsheet software to manipulate the data as described earlier. The second is to use software specifically designed for pavement management. Both methods are effective. A list is provided in the manual of popular spreadsheets and the hardware on which they operate. Most of these software packages are priced between $100 and $500.

The spreadsheet is basically a large table with rows and columns of cells. The computer user can place a number, a label, or a formula in each cell. If the cell entry is a formula, it is defined as a function of the current values in certain other cells. The spreadsheet can calculate new values for each function as the input values of the table change.

A brief example of a pavement management spreadsheet is shown in Figure 10. Each row corresponds to a street segment and each column to data or results of the program. A key at the bottom indicates the meanings of the codes for surface type, traffic, and so forth. Once the data are entered as shown, unit costs for each type of maintenance can be entered for the six traffic and treatment categories and automatically assigned to street segments. The computer can then multiply these unit costs by street length to derive project costs.

The remainder of the programming could be very time-consuming if done by hand, but is trivial on the computer. Projects can be sorted by year of treatment, surface type, treatment, or any other category desired. If some projects must be deferred to a later year, the entire process can then be easily repeated after the recommended treatment and year are revised.

The second option is to use a data base manager tailored to the pavement management process. A list of pavement management software encountered in this study is shown in Figure 11. These powerful programs are capable of handling large data bases and producing useful statistics and graphics.

CASE STUDIES

The experiences of five communities were reported in the manual, and each took a very different approach to the

HILTON
Pavement Management Survey
cem 07-10-86

STREET NAME	SURFACE TYPE	LENGTH	WIDTH	TRAFFIC	CONDITION	RECOMM. TREAT.	YEAR	UNIT COST	TOTAL COST
Monument St.	1	0.41	30	3	3	2	4	$129,000	$52,890
Linden St.	1	0.07	28	1	2	2	1	98,000	6,860
Rubbly Rd.	1	0.28	26	1	2	1	1	39,000	10,920
Hill Top Dr.	1	0.70	25	2	1	2	1	109,000	76,300
Bruce Lane	1	0.16	30	1	4	1	7	66,000	10,560
Porter St.	1	0.43	20	2	2	1	4	66,000	28,380
Grapevine Rd.	1	0.61	24	3	4	1	3	86,000	52,460

KEY:

Surface Type:
1. Bituminous pavement
2. Aggregate

Traffic:
1. Low
2. Medium
3. High

Condition:
1. F Failure
2. E Very Poor
3. D Poor
4. C Fair
5. B Good
6. A Excellent

Recommended Treatment:
1. Resurface
2. Reconstruct

Unit Costs (per mile):

	Resurfacing	Reconstruction
Low Traffic	$39,000	$98,000
Medium Traffic	66,000	109,000
Heavy Traffic	86,000	129,000

FIGURE 10 Sample of pavement management spreadsheet.

Name of Software	Type of System	Operating System	Supporting Software	Phys. Storage	Mass Data Required	Other Hardware	Data I/O	Cost	Date	Ref.	Comments
COMPAVE	Radio Shack Model 16	TRS-DOS	Radio Shack BASIC	64k	2 floppies		interact. prn	$7,500	10/85	Allan Davis	
IMS	IBM-pc & compat.	MS-DOS	RUN-TIME D-Base (incl.)	256k	1 floppy 1-10MB hard disk	132colprn	interact. prn graphics	$1,995 SW + modif.	10/85	Gordon Derring	
PAVER	see comments	Control Data	Comm. SW for PC	128 to 256k	2 floppies	Mainframe Computer	file, prn graphics	approx. $3-6/SY	10/85	Mike Hill	Entryof field data with PC, to Mainframe Output to PC
PMI	most Micros	MS-DOS CP/M	RUN-TIME D-Base (incl.)	64k min.	1 floppy 1-10MB hard disk		interact. prn	$3,000 $6,000 $9,000	10/85	Harris & Assoc. Texas A&M	Costs are Econ., Reg., & Deluxe
PMS	most Micros	MS-DOS CP/M	RUN-TIME DATAFLEX (incl.)	256k	1 floppy 1-10MB hard disk		interact. prn	$1,500 to $11,000	10/85	Vanasse-Hangen	
FPMS	IBM-pc	MS-DOS	Advanced BASIC	256k	1 floppy		interact. prn	Public Domain	10/85	CALTRANS	
STAMPP PMS	IBM-pc & compat.	MS-DOS	Advanced BASIC	128k	2 floppies min.		interact. prn	Public Domain	10/85	Penn.DOT	

FIGURE 11 Pavement management software (*6*).

problem. One survey was very simple and gathered only the following information for each street:

- Street name,
- Segment,
- Length and average width,
- Surface type and area in yd^2, and
- Condition and recommended action.

This information was sufficient to assemble a 3-year program of rehabilitation. The total cost of this effort, including field work and record-searching, was about six person-weeks of staff time. The result was a documented list of needs for the annual budget proposal. The next step for this community is to program each road into a long-range plan to make the costs of deferring maintenance obvious.

Another community that evaluated private roads developed several innovative factors to measure the significance of each road and its condition:

Housing Factor—This factor indicates service to homes:

1—Fifteen houses or less
2—More than 15 but not exceeding 30
3—More than 30 but not exceeding 45
4—More than 45

Artery Factor—This factor indicates the road function:

1—Minor Residential — Road provides access to houses primarily on that street.
2—Residential Collector — Road feeds into "subdivision" providing primary access to houses on other streets.
3—Thru Connector — Road serves as primary connector between two major roads.

Surface Factor—This factor indicates the road surface condition:

1—Very Good — Road surface generally smooth, can travel at legal speed without damage or loss of control.
2—Good — Road surface somewhat rough, can travel at legal speed with moderate care.
3—Fair — Road surface rough in many locations, can travel at slightly below legal speed with moderate care.
4—Poor — Road surface rough in many locations, can travel only at speeds substantially below legal limit.
5—Very Poor — Road surface very rough throughout, travel on road must be very slow and erratic to avoid damage or loss of control.

The five case studies presented in the manual were for road systems that varied in size from 33 miles to over 100 miles. Although three studies were completed by town highway department staff, two were performed by consultants (for the largest and smallest of the road systems). None used automated road survey equipment, but the consultant studies used a microcomputer. The methodologies of these studies varied widely, but in each case the needs of the road system were clarified.

CONCLUSION

Pavement management is effective in both reducing road maintenance costs and improving road conditions. Although resources for planning low-volume roads may be limited, pavement management is important to these facilities. The simplified methods presented make it possible for any road maintenance agency to implement a system of pavement management programming. The goal for that system is to document road maintenance needs to the point at which the costs of deferring maintenance are clear.

Some jurisdictions may still confront the obstacle that the funds needed for the programmed maintenance projects are simply not available. The Metropolitan Area Planning Council is currently researching this problem. By comparing local estimates of need with statistics on recent expenditures, the MAPC will attempt to ascertain whether there is a shortfall and, if so, to quantify it at the regional level for a group of 101 communities. Alternate sources of funding will be explored.

ACKNOWLEDGMENTS

The MAPC project team is very grateful for the participation and contributions of the many professional groups, trade organizations, government agencies, and consultants who provided a wealth of literature and helpful advice. Several communities took the manual to the field and tested its methods. The Technical Advisory Committee also provided invaluable guidance in the identification of resources and methods and in directing the best response to the programming needs of our communities.

The following people served on the Technical Advisory Committee: Peter Burnham, Superintendent of Streets, Wenham; Tony Celli, Director of Public Works, Walpole; John Collura, Professor of Civil Engineering, University of Massachusetts at Amherst; Lawrence DeCelle, Director of Public Works, Milton; Lewis Edgers, Professor of Civil Engineering, Tufts University; Ken Garrity, Edwards & Kelcey, Boston; Joe Hegarty, Highway Maintenance Engineer, MDPW, Boston; John Schoon, Professor of Civil Engineering, Northeastern University; Herbert Simmons, Director of Public Works, Hanover; Matt Turo, Assistant Research Coordinator, Research and Materials, MDPW, Wellesley; and Bob Patneaude, MAPC Liaison, MDPW.

Publication of *Pavement Management: A Manual for Communities* was made possible by funding from the Federal Highway Administration and the Massachusetts Department of Public Works.

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An Analysis of the Condition of Gravel and Stone Roads in Indiana

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Condition relationships for gravel and stone roads were determined using road condition variables of roughness number, average rut depth, and Clegg Impact Value (a measure of in situ pavement strength). The studies were undertaken in Bartholomew, Huntington, Jasper, Tippecanoe, and Warrick Counties in Indiana. Roughness was measured at 20 mph (32 kph) over 1-mi test sections using a PCA roadmeter from the Indiana Department of Highways, Division of Research and Training. The independent variables of roadway geometry, material, and use characteristics in the regression analysis included average road width; percentage of surface camber; terrain (indicator or dummy variable); surface material parameters [P200, P40, P10, D95 (in), fineness modulus, liquid limit, and plasticity index]; and average daily traffic volume. In each 1-mi section, one surface material sample was collected for testing and other measurements were taken at 0.1-, 0.3-, 0.5-, 0.7-, and 0.9-mi locations. Most correlation coefficients were considered significant at α = 0.1 during the regression analysis. After the covariance and variance were analyzed, suitable equations were determined for each of the three condition variables. Two equations, including and excluding fineness modulus as a measure of coarseness of surface material, were determined. The coefficients of the determination of the six equations ranged from 0.52 to 0.69. The regression analysis, combined with degree of corrugation, strength, and surface rutting, provided a basis for the evaluation of the performance and maintenance management of low-volume gravel and stone roads in Indiana.

In the state of Indiana, as in many other countries with similar levels of development, gravel and stone roads are predominant in local highway networks. County and other local highway agencies, often with limited resources, are responsible for their maintenance. Road condition is usually assessed during routine or special road inspections by applying a visual assessment and personal judgment to recommend maintenance or major improvements. The condition of gravel or stone roads changes rapidly and is usually affected by a number of factors including road materials, environmental effects, traffic volume, and maintenance. An understanding of these factors can help establish suitable gravel road maintenance levels and simplified procedures for local highway agencies.

Relationships are presented between the road condition variables of roughness number, average rut depth, Clegg Impact Value, and unpaved road surface material and other roadway variables. The measurements that were taken of corrugations and the material characteristics of corrugated, weak, and rutted surfaces are also discussed to provide further insight into the factors that affect the performance and maintenance of gravel and stone roads.

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STUDY LOCATION AND EXPERIMENTAL DESIGN

Unpaved roads of a variable total length were selected from five counties in Indiana (Figure 1). The statistical experimental design adopted was a nested factorial design with unequal cells. Roads were nested in counties and sections were nested within roads. The unequal cells resulted from the inability of counties to select the same number and length of roads (Table 1).

VARIABLES ANALYZED

The three dependent variables that described road condition in the regression analyses were as follows:

- Roughness number (RN) measured in counts per mile using a PCA roadmeter at a measuring speed of 20 mph (32 kph),
- Average rut depth (ARD) measured in inches from surface to bottom of rut, and
- Clegg Impact Value (CIV), a measure of pavement strength using the Clegg Impact Tester (CIT).

The latter two variables were also analyzed as dependent variables. The independent variables used in this study are shown in Table 2 together with the limits of values that were measured and used in later equations. Previous research on unpaved roads also used most of these variables (*1-5*).

CONSIDERATIONS FOR FIELD MEASUREMENTS

Roughness

A PCA roadmeter supplied by the Division of Research and Training (DRT) of the Indiana Department of Highways (IDOH) was used to assess unpaved road roughness by divisional staff. The PCA roadmeter was chosen over the Mays ridemeter because of frequent cable failure on the Mays meter during initial trials. Roughness was measured at a speed of 20 mph (32 kph). This speed was selected because it reduced these cable problems. A speed of 50 mph (80 kph) was normally used for roughness measurements on paved roads using the PCA roadmeter. Roughness was measured over an entire 1-mi (1.6-km) section following the normal practice by the DRT of

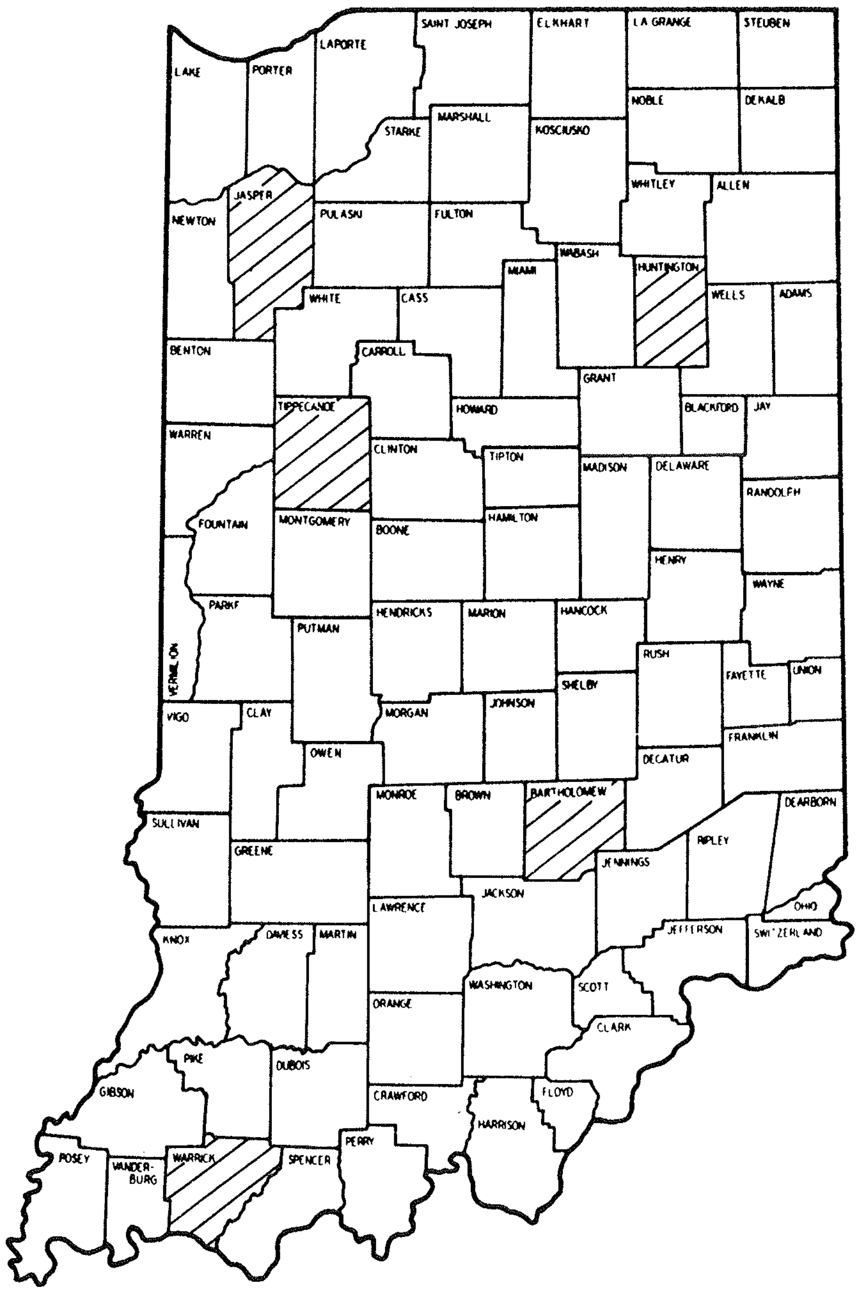

FIGURE 1 Location of study counties in the state of Indiana.

measuring roughness on 1-mi sections of the state highway network. Instrument calibration was provided from measurements by the DRT on an existing paved road section and other measurements that related roughness at various measuring speeds and measurements on the gravel road sections (*6*). However, roughness measured in counts per mile was not related to units such as inch per mile or quarter-car index that were used in other studies (*1, 4, 5, 7*).

Terrain, CIV, Rut Depth, and Roadway Characteristics

The effect of road vertical gradient and alignment was considered by applying a dummy or indicator variable (G) in the regression relationships (Table 2). CIV, rut depth, and other roadway characteristics, such as road width and camber, were measured at 0.1- , 0.3- , 0.5- , 0.7- , and 0.9-mi locations on each section (*6*). The CIV at each cross-section was measured at the centerline and transversely every 2.5 ft for the entire cross-sectional width. This procedure was first used in previous research at Purdue University in which the highest CIV readings occurred about 2.5 ft from the centerline (*8, 9*). The potential full range of CIV values on the cross-section was included in calculating average CIV by using 2.5 ft intervals.

Road Surface Materials Sampling and Testing

A representative sample of road surface material was obtained from each section down to a depth of 4 in (10 cm) from the

TABLE 1 DISTRIBUTION OF COUNTY ROADS AND SECTIONS IN REGRESSION ANALYSIS

County	Road Number	Number of Sections
Bartholomew	1	3
	2	1
	3	1
Huntington	1	1
	2	1
	3	1
	4	1
	5	1
	6	3
	7	2
	8	2
Jasper	1	1
	2	2
	3	1
	4	1
	5	2
	6	2
	7	2
Tippecanoe	1	2
	2	2
	3	2
	4	2
	5	1
	6	1
	7	2
Warrick	1	1
	2	1
	3	1
	4	2
	5	2
	6	2

surface. Additional samples were obtained from locations with identifiable corrugations or weak and deeply rutted surfaces. As a result, material characteristics of corrugated and weak spots were determined.

Gradation and liquid and plastic limits were determined for each sample. However, all of the surface gravel samples tested were found to be nonplastic; therefore, only the liquid limit was used in the regression analysis.

STATISTICAL ANALYSIS

Relationships for three dependent variables (*RN, ARD,* and *CIV*) were determined by applying analysis of variance or covariance followed by multiple regression analysis. The results from the analysis showed that the "county" factor was not significant for the dependent variables of roughness and average rut depth but was significant for CIV. As a result, the data for all five counties were combined for roughness and average rut depth and "county" was applied as a dummy variable in the regression equation for CIV (*6*). The coefficient of multiple determination, R^2, and the adjusted coefficient of determination, R_a^2, are presented for all the regression equations. Because the R^2 value usually increases with the number of independent variables in the equation, the R_a^2 value given in Equation 1 accounts for the effect of the number of independent variables in the equation (*10*).

TABLE 2 VARIABLES MEASURED OR CALCULATED FOR USE IN THE ANALYSES

Variable	Limits on Value in the Analysis	Description
1. Road width, (W ft) (Independent Var.)	15.0 - 26.8	Average section unpaved roadway width.
2. Surface Camber (CA %) (Independent Var.)	1.1 - 5.6	Average crossfall in the section.
3. Grade or Terrain (G) (Dummy Var.)	0 or 1	Gradient/Terrain Flat/Rolling = 0 Rolling to Hilly = 1
4. Soil parameters Surface gravel		(Independent Vars.)
(i) P#200 (%)	3.0 - 18.0	Percent passing ASTM #200 sieve (.075mm)
(ii) P#40 (%)	11.0 - 35.0	Percent passing ASTM #40 sieve (.425mm)
(iii) P#10 (%)	15.0 - 71.0	Percent passing ASTM #10 sieve (2.0mm)
(iv) D95 (in)	0.35 - 2.5	Equivalent sieve size passing 95% of the material.
(v) FM	3.05 - 6.7	Fineness Modulus - sum of cumulative percent retained on sieve sizes - 3/4 in. to #100, divided by 100.
(v) LL	4.8 - 20.0	Liquid Limit
(vi) PI	Non Plastic	Plasticity Index
5. ARD (in) (Dependent/Independent)	0.0 - 1.2	Average Rut Depth
6. CIV (Dependent/Independent)	24.9 - 73.1	Clegg Impact Value
7. Average Daily Traffic (ADT) (Independent Variable)	14 - 282	Average Daily Traffic Volume.
8. Roughness Number (Idependent Variable)	246 - 5903	Roughness in Counts/Mile at 20 mph (32 kph)

$$R_a^2 = 1 - \frac{(n - 1)}{(n - p)} (1 - R^2) \qquad (1)$$

where

n = number of observations in the sample,
p = number of parameters, and
R^2 = coefficient of multiple determination determined from the least-squares procedure.

A correlation analysis between pairs of variables was undertaken to determine the significant independent variables for multiple regression analysis. Correlation coefficients for *RN*, *ARD*, *CIV*, and each independent variable, as well as their level of significance and the corresponding 95 percent confidence interval, are shown in Table 3. Variables with population correlation coefficients significant at α levels of less than 0.10 were included in the regression analyses.

Roughness Number

The negative correlation coefficients obtained imply that an increase in CIV, the percentage of surface gravel or stone passing a No. 40 (.425-mm) sieve (P40) and a No. 10 (2.0-mm) sieve (P10) is associated with lower roughness numbers. A higher CIV or higher in situ strength is logically related to a higher density. As a result, it could be concluded that well-compacted gravel or stone road surfaces would exhibit lower roughness levels. The positive correlation coefficients imply that gravel roads with deeper ruts, rolling to hilly vertical alignment (*G*), larger sieve size passing 95 percent of material (D95), higher fineness modulus (or coarseness), higher liquid limit, and higher ADT would tend to have surfaces with higher roughness numbers.

Equations 2 and 3 are the regression relationships obtained for surface roughness. Equation 3 uses the fineness modulus (FM) to describe surface material coarseness in place of variables P10, P40, and D95. All variables used in Equations 2 to 7 have been previously defined in Table 2.

TABLE 3 RESULTS OF CORRELATION ANALYSIS FOR ROUGHNESS, AVERAGE RUT DEPTH, AND CLEGG IMPACT VALUE RELATIONSHIPS

Variable	Correlation Coefficients		
	Roughness	Rut Depth	CIV
Average Rut Depth	.453* (.001)[1] .197/.651***	-- -- --	-- -- --
CIV	-.383* (.039) -.600/-.114	.007 (.371) -.275/.288	-- -- --
Gradient or Terrain (Dummy Var.)	.254* (.039) -.029/.500	.048 (.371) -.236/.325	.059 (.344) -.226/.335
Road Width	.179 (.110) -.108/.438	-.093 (.263) -.365/.193	.221** (.063) -.064/.473
Camber (%)	-.057 (.348) -.333/.228	.046 (.377) -.238/.323	-.096 (.255) -.367/.190
P#200	-.115 (.216) -.384/.172	.123 (.199) -.164/.391	.525* (.001) .286/.702
P#40	-.290* (.022) -.528/-.010	-.404* (.002) -.615/-.139	.063 (.335) -.222/.338
P#10	-.597* (.001) -.752/-.380	-.642* (.001) -.782/-.440	.080 (.292) -.206/.353
D95	.495* (.001) .248/.681	.682* (.001) .496/.808	.080 (.432) -.206/.353
FM	.589* (.001) .369/.747	.666* (.001) .473/.798	-.008 (.479) -.289/.274
Liquid Limit	.190** (.096) -.096/.447	.315* (.014) .037/.548	.076 (.303) -.210/.350
ADT	.217** (.067) -.068/.470	.151 (.151) -.136/.415	.081 (.291) -.205/.354

NOTES: 1. Numbers in parentheses denote significance level
* Coefficients significant at α = .05
** Coefficients significant at α = .10
*** 95 percent confidence limits for the coefficient.

$$RN = 4595.23 - 66.03\ P10 - 37.35\ CIV + 7.36\ ADT + 603.5\ G - 89.62\ LL + 85.02\ P40 + 880.00\ D95 \quad (2)$$

$$R^2 = 0.649 \qquad R_a^2 = 0.589$$

$$RN = -547.87 + 1300.94\ FM - 60.60\ CIV + 8.24\ ADT + 633.08\ G + 114.09\ P200 - 155.53\ LL \quad (3)$$

$$R^2 = 0.686 \qquad R_a^2 = 0.641$$

The relative significance of the added contribution of each variable to the total R^2 is shown in Table 4. All of the variables, significant at $\alpha = 0.10$, in both equations make contributions to the R^2.

Average Rut Depth

From the correlation analysis, ARD showed a negative correlation with P40 and P10. This implies that larger quantities of smaller coarse sizes (P40 and P10) and the sand sizes contribute to decreasing average rut depth. On the contrary, ARD tends to increase with increasing coarse sizes as represented by D95, higher values of fineness modulus that represent an overall coarseness, and increasing liquid limit. The tendency to rut more with coarser material is usually the result of instability in the material created by lack of adequate soil binder or smaller aggregate sizes that provide a keying function. Surface material deficient in smaller sizes is usually less dense and easily dispersed by vehicles. Coarser surfacing materials facilitate surface moisture penetration and also retain moisture for longer periods of time. The moisture retained would affect finer, moisture-susceptible materials underneath the coarser surfacing, which would tend to rut under traffic. Although ADT was not significant at $\alpha = 0.10$, the low positive correlation shows only a slight tendency for ARD to increase or decrease with traffic volume.

Equations 4 and 5 describe relationships for ARD obtained from multiple regression analyses. In Equation 5, the fineness modulus replaces variables D95, P10, and P40 to describe the coarseness of the surface material.

$$ARD = -.018 + 0.457\ D95 + .014\ P200 - .010\ P10 + 0.001\ ADT + 0.008\ P40 - .009\ LL - .02\ G \quad (4)$$

$$R^2 = 0.577 \qquad R_a^2 = 0.504$$

$$ARD = -1.315 + 0.358\ FM + 0.019\ P200 + 0.001\ ADT - 0.0162\ LL - 0.039\ G \quad (5)$$

$$R^2 = 0.517 \qquad R_a^2 = 0.46$$

The significance of the contribution to R^2 made by the addition of each variable is shown in Table 5. In Equation 4, variables *D95, P200,* and *P10* make contributions to the R^2 significant at $\alpha = 0.10$ and explain up to 54.5 percent of the R^2. For Equation 5, variables *D95, P200,* and *ADT* explain about 54.1 percent of R^2. The rut depth values measured in the field were generally low; several locations registered zero rut depths. Therefore, average rut depth can only be predicted within a narrow range of values obtained in the field.

TABLE 4 CONTRIBUTIONS TO R^2 VALUE BY VARIABLES IN THE ROUGHNESS EQUATIONS

Independent Variables in Equation 2					
Step	Variable	R	R^2	R^2 Change	Significance
1	P#10	.5968	.3562	.3562	.000
2	CIV	.6849	.4691	.1129	.003
3	ADT	.7146	.5106	.0415	.057
4	G	.7406	.5485	.0379	.061
5	LL	.7636	.5830	.0345	.066
6	P#40	.7875	.6202	.0372	.049
7	D95	.8055	.6489	.0286	.075
Independent Variables in Equation 3					
Step	Variable	R	R^2	R^2 Change	Significance
1	FM	.5887	.3466	.3466	.000
2	CIV	.6998	.4897	.1431	.001
3	ADT	.7383	.5451	.0554	.024
4	G	.7608	.5788	.0338	.067
5	P#200	.7821	.6116	.0328	.063
6	LL	.8282	.6860	.0743	.003

TABLE 5 CONTRIBUTIONS TO R^2 VALUE BY VARIABLES IN THE AVERAGE RUT DEPTH EQUATIONS

Independent Variables in Equation 4					
Step	Variable	R	R^2	R^2 Change	Significance
1	D95	.6820	.4652	.4652	.0
2	P#200	.7168	.5138	.04862	.037
3	P#10	.7380	.5447	.0309	.087
4	ADT	.7511	.5641	.0195	.168
5	P#40	.7563	.5721	.0079	.377
6	LL	.7587	.5757	.0036	.553
7	G	.7593	.5765	.0008	.781
Independent Variables in Equation 5					
Step	Variable	R	R^2	R^2 Change	Significance
1	D95	.6820	.4652	.4652	.0
2	P#200	.7168	.5138	.0486	.037
3	ADT	.7356	.5412	.0274	.108
4	FM	.7444	.5541	.0130	.264
5	LL	.7466	.5574	.0033	.577
6	G	.7474	.5586	.0012	.743

Clegg Impact Value

The percentage of materials passing a No. 200 (.075-mm) sieve (P200) and the average road width, *W*, were two variables that had correlation coefficients significant at $\alpha = 0.10$ with CIV. This implies that gravel road sections with wider roadways and roads with a higher proportion of fines (P200) would usually exhibit higher CIVs. The latter is generally true because sufficient fines are required to obtain denser material with a higher CIV when compacted.

Although CIV did not show a significant correlation with ADT, average road width had a positive correlation of 0.22 with ADT significant at $\alpha = 0.06$. This level of correlation is only a slight indication that roads with higher ADT are wider. Because most gravel and stone roads in Indiana are not usually rolled after regraveling, but are allowed to be compacted by traffic, the wider roads with potentially higher traffic volumes may exhibit higher CIVs. This factor, however, requires further investigation.

Because "county" was significant at $\alpha = 0.05$ in the analysis of covariance, dummy or indicator variables Z1, Z2, Z3, and Z4 were introduced into the regression. The relationships derived for CIV are presented in the following equations.

$$CIV = 20.203 + 8.351\ Z1 - 14.345\ Z2 - 12.330\ Z3 - 1.359\ W - 5.645\ Z4 + 0.143\ P200 \quad (6)$$

$$R^2 = 0.681 \qquad R_a^{\ 2} = 0.635$$

$$CIV = -2.235 + 14.506\ Z1 + 4.452\ FM + 0.604\ P200 - 1.003\ W - 7.960\ Z2 - 4.875\ Z4 - 4.071\ Z3 + .003\ ADT \quad (7)$$

$$R^2 = 0.694 \qquad R_a^{\ 2} = 0.633$$

The significance of contributions made by the various independent variables to the R^2 obtained for each equation is shown in Table 6. In Equation 6, apart from county variables, road width makes a significant contribution to the R^2coefficient. However, in Equation 7, the fineness modulus (*FM*), *P200*, and road width (*W*) make a total contribution to R^2 of 68.4 percent if county variable *Z2* is included. Both equations are, however, applicable to the study areas only because the "county" variables are used.

DISCUSSION

Grading

In previous research in Kenya and Brazil, the number of days since the last grading or the cumulative traffic volume since the last grading were used as independent variables (*1, 2, 4, 5*). In this research, little variation was found in the data obtained to determine the number of days since last grading in any county. The differences between counties did not significantly affect relationships with road condition variables also; the grading variable was therefore omitted.

General county maintenance practice has been to avoid grading when the surface crust is well developed. Some study roads had not been graded for over 70 days and no significant effects on road condition were detected. Nevertheless, evidence from one road section that was graded the day of the measurements showed substantial reductions in roughness with grading. The average roughness on a lane in the same direction on four, unbladed 1-mi sections was 1,944 counts/mi compared to 741 on the opposite bladed lane, which is a 62 percent

TABLE 6 CONTRIBUTIONS TO R^2 VALUE BY VARIABLES IN THE CLEGG IMPACT VALUE EQUATIONS

Independent Variables in Equation 6					
Step	Variable	R	R^2	R^2 Change	Significance
1	z1	.6927	.4798	.4798	.0
3	z2	.7551	.5701	.0864	.002
4	z3	.7839	.6145	.0444	.029
5	W	.8166	.6668	.0523	.012
6	z4	.8243	.6795	.0128	.205
7	P#200	.8250	.6806	.0011	.708

Independent Variables in Equation 7					
Step	Variable	R	R^2	R^2 Change	Significance
1	z1	3.3511	.6927	.4798	.0
2	FM	.7743	.5996	.1198	.001
3	P#200	.8023	.6436	.0440	.023
4	W	.8156	.6652	.0216	.100
5	z2	.8276	.6849	.0198	.108
6	z4	.8317	.6917	.0068	.341
7	z3	.8331	.6940	.0023	.581
8	ADT	.8332	.6942	.0001	.898

reduction. Roughness on another 1-mi (1.6-km) section dropped by 52 percent to 1,076 counts/mi from 2,246 counts/mi after grading. On a 1-mi section, half of which had been graded, the average reduction in roughness was 24 percent from 1,974 to 1,501 counts/mi. This indicates that on the day of grading, roughness values are likely to fall by about 50 percent on a typical gravel or stone road.

These observations confirm results published by Carmichael et al. for similar measurements on gravel roads in Bolivia (*11*). During the dry season in Bolivia, which is similar to summer conditions in Indiana, same-day reductions in roughness measured with the Mays ridemeter were between 41 percent and 52 percent. In the wet season in Bolivia, however, same-day reductions were reported between 1 and 38 percent. Roughness generally returned to original levels, or higher, about 20 days after grading in Bolivia.

Surface Material Characteristics

In the absence of a maintenance variable such as grading, surface material characteristics of gradation and liquid limit explain between 35 and 52 percent of the variability in the road condition variables of roughness, average rut depth, and pavement strength (CIV). Average material characteristics identified on roads in the five counties studied are presented in Table 7 and Figures 2 to 6.

Gradation specifications for No. 53 and No. 73 crushed stone that are recommended for base construction and gravel or stone road surface courses in Indiana are also shown in Figures 2 to 6. Indiana specifications state that in addition to the gradation band, the fraction passing a No. 200 sieve should not exceed two-thirds of the fraction passing the No. 30 sieve. Liquid limit should not exceed 25 (35 if slag) and the plasticity index should not exceed 5. When used as unsurfaced gravel or stone base, the amount passing a No. 200 sieve should be between 5 and 12 percent and the plasticity index should not exceed 7. Although not stated specifically, the grading ranges limit top sizes of the required material to less than 1.5 in. for No. 53 stone or aggregate and less than 1 in for No. 73.

Indiana counties have often adopted less stringent material specifications for gravel and stone roads for economic reasons. The study counties appear to have differing practices regarding material used on unpaved roads (Table 7 and Figures 2 to 6). Some counties allow maximum aggregate sizes greater than 2 or 3 in, whereas other counties have adopted fine gradation requirements. However, in some cases the finer gradation is partly a result of the effect of traffic abrasion.

The facility with which a surfacing material develops a crust appears to be important to the amount of roughness. The lowest average roughness value of 959 counts/mi was measured on unpaved roads in Huntington County followed by 2,019 and 2,819 counts/mi on roads in Tippecanoe and Jasper counties, respectively. Using the CIV as a relative measure of crust development, Huntington County roads exhibited the highest average CIV of 57, whereas Tippecanoe and Jasper counties had the lowest average values of 35 and 33, respectively. It is likely that the well-developed surface crust found on most gravel roads in Huntington County contributed to the better road performance compared to the other four counties. Huntington and Tippecanoe counties apply crushed limestone of No. 53 or No. 73 specifications to unpaved road surfaces.

TABLE 7 SUMMARY OF AVERAGE COUNTY ROAD SURFACE GRAVEL AND CROSS-SECTIONAL CHARACTERISTICS

Variable Description	Material Property or Cross-Sectional Characteristic by Study County				
	Bartholomew	Huntington	Jasper	Tippecanoe	Warrick
Number of Sections of Road	5	12	12	12	9
Fineness Modulus	4.4	3.5	4.0	3.9	4.9
Liquid Limit (Range)	13 (12.5-14)	13 (12-14)	11 (7-15)	12 (7-17)	17 (14-20)
Roughness (Counts/Mile)	2858	958	2819	2012	3369
Roadway Width (in.)	16.3 (0.9)[1]	19.2 (1.5)	19.1 (2.3)	19.1 (1.6)	20.4 (3.8)
Camber (%) (Range)	3.4 (.7)[1] (2.3-4.7)	2.8 (.9) (1.4-4.7)	2.8 (.9) (.8-4.1)	4.5 (1) (2-6)	3.84 (1.4) (.6-6.2)

NOTES

1. Numbers in parentheses are standard deviation values

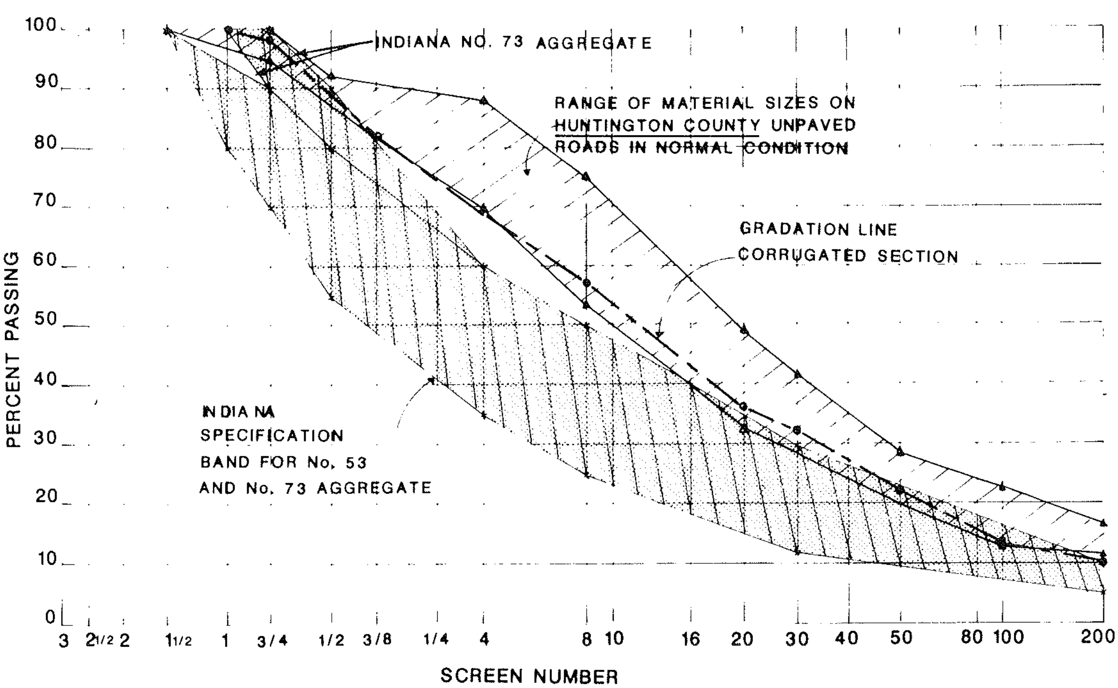

FIGURE 2 Surface gravel gradation characteristics in Huntington County.

Jasper County practiced the application of much coarser No. 5 crushed stone after the spring thaw to weakened areas and No. 73 aggregate at other times.

Both Bartholomew and Warrick counties in the southern part of the state use materials that are, on the whole, coarser than Indiana No. 53 or No. 73 aggregate. Warrick County applies No. 5 crushed limestone (Figure 5), whereas Bartholomew County applies No. 8 or No. 9 crushed limestone (Figure 6). Roads in those two counties had the highest average roughness of 2,858 and 3,369 counts/mi, respectively. The average CIV in both counties was 44. In Warrick County, some existing roads had been reconstructed as part of coal mining operations using coarse crushed rock.

No materials tested had a liquid limit above the recommended upper limit of 25 (35 for slag) (*12*). In order to gain further insight on road condition effects, surface material and other characteristics of corrugated, weak, and rutted road surfaces are discussed in the following sections.

Characteristics of Weakened and Rutted Sections

Gradation characteristics of four weak surface areas identified on two roads in Jasper County (250E and Division Road) and two roads in Tippecanoe County (850W and 1300S) are presented in Figures 3 and 4, respectively. The CIV values

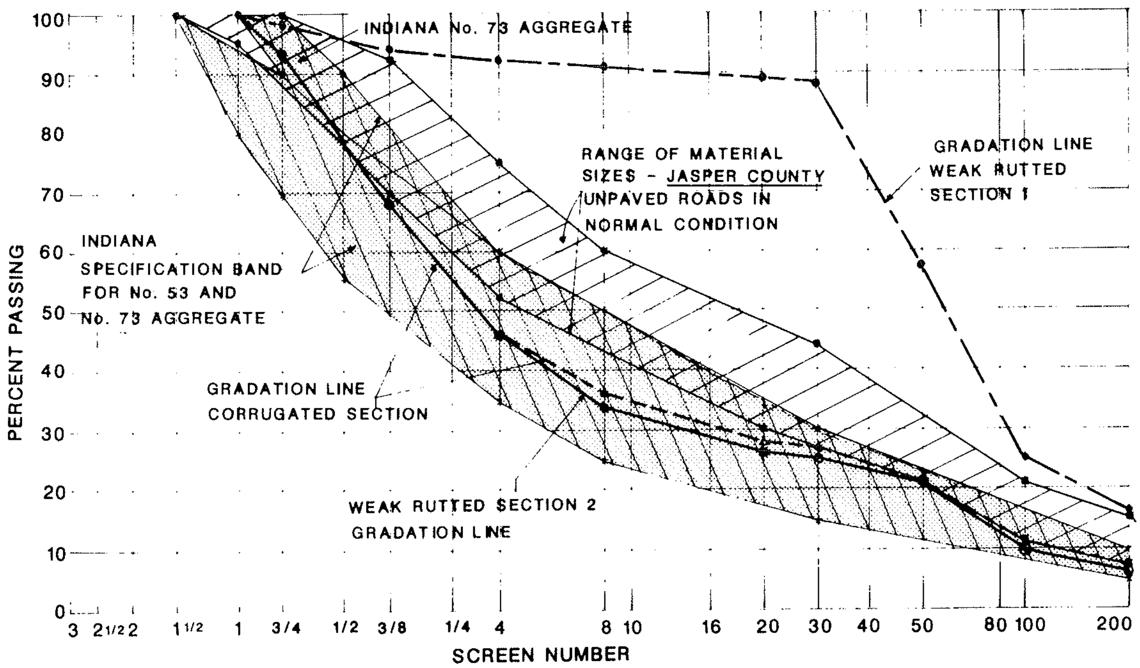

FIGURE 3 Surface gravel gradation characteristics in Jasper County.

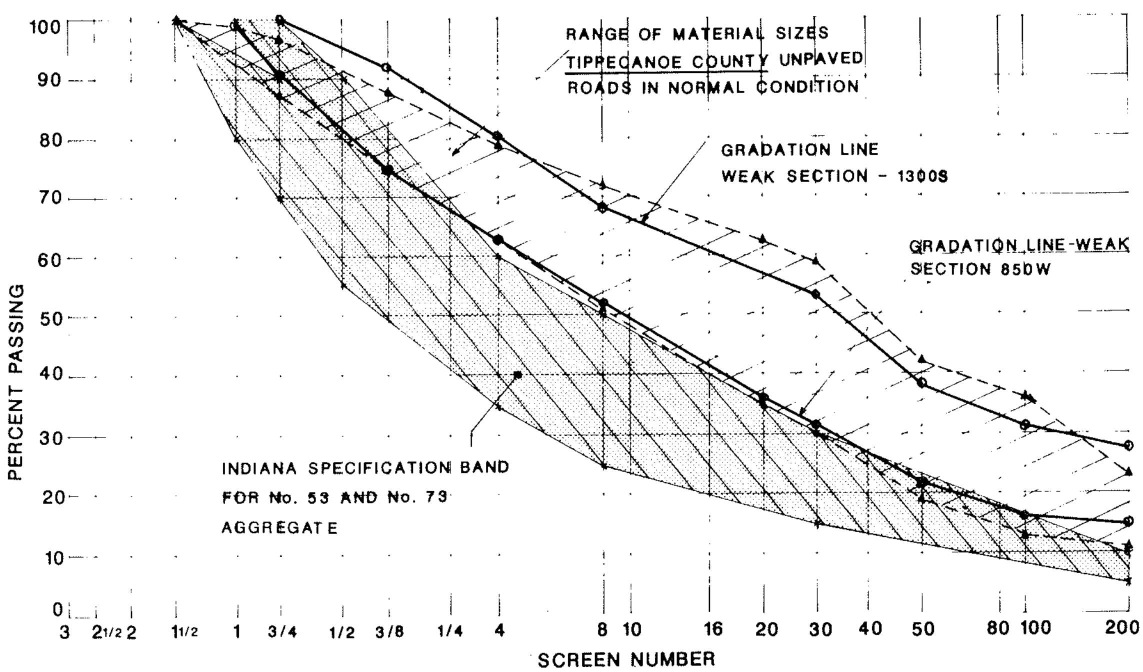

FIGURE 4 Surface gravel gradation characteristics in Tippecanoe County.

measured on these sections were 17, 25, 8, and 10, respectively. These CIV values were much lower than values measured on any road section that performed satisfactorily.

Weakened and rutting areas can also be identified by other characteristics. In the case of one road in Jasper County and another in Tippecanoe County, the surface material was loose and coarser than the county average. The surface exhibited a low CIV and rutting. Little compaction had taken place and the surfaces were prone to failure.

Weak areas can develop in cases in which the surface material is finer than average and is therefore sensitive to moisture. Loss of surfacing as a result of inadequate crust development can also expose a fine-grain subgrade to the detrimental effects of moisture. Moisture-sensitive subgrades combined with poor drainage likewise create weakened conditions.

Characteristics of Corrugated Surfaces

Corrugation or washboarding is a common type of gravel road distress; therefore, an understanding of its formation is essential in gravel road maintenance. Material gradation lines are

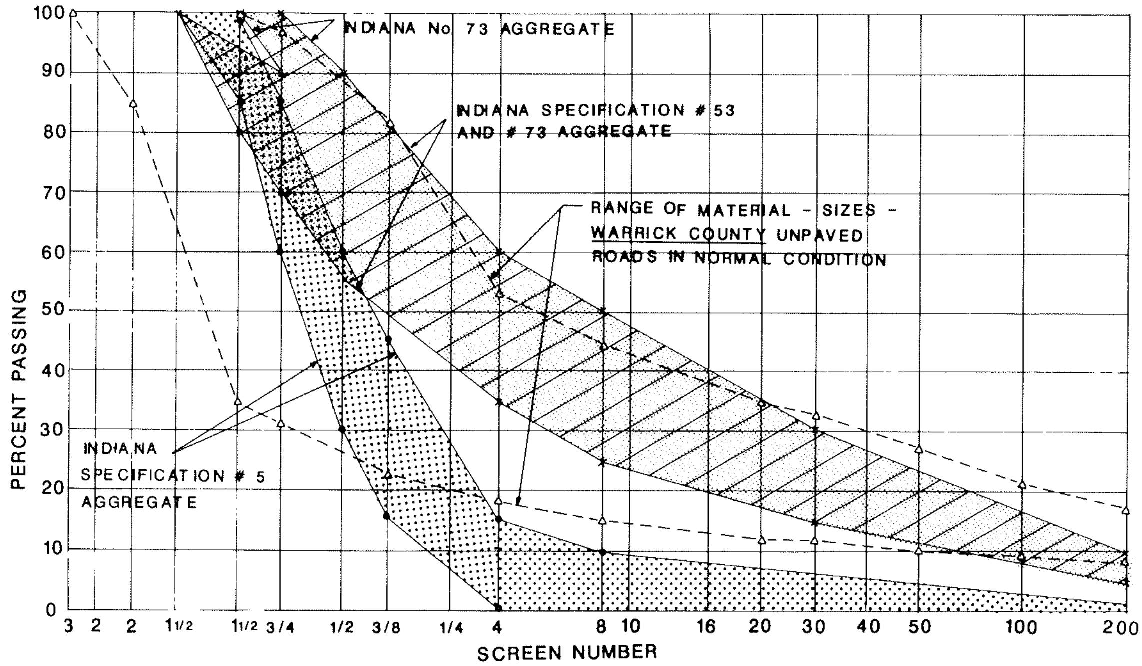

FIGURE 5 Surface gravel graduation characteristics in Warrick County.

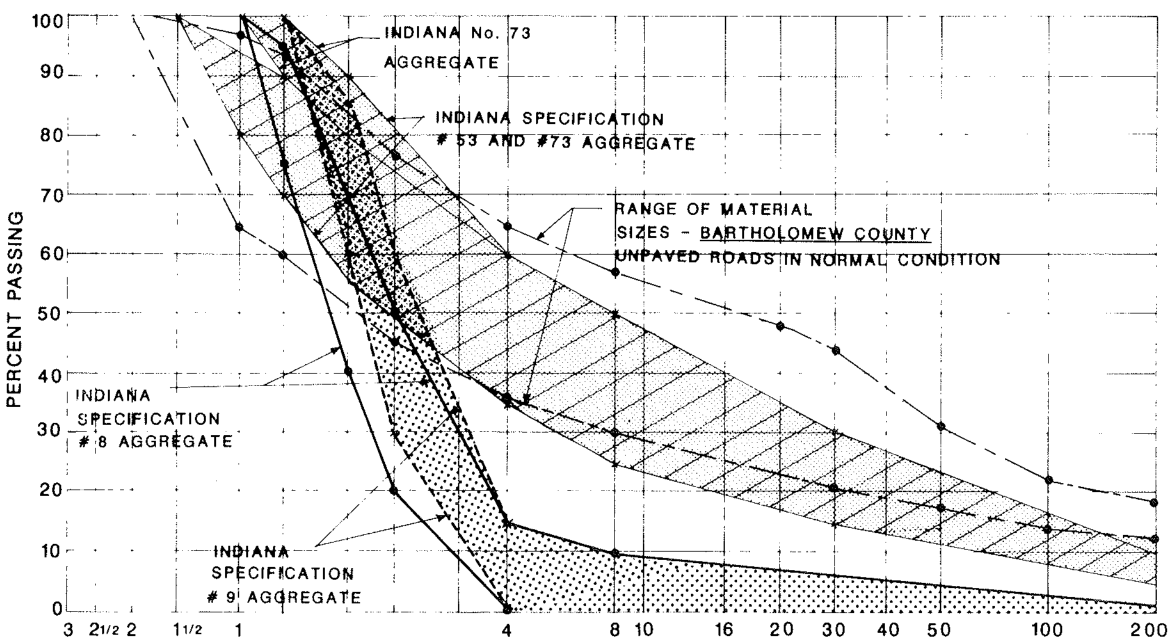

FIGURE 6 Surface gravel gradation characteristics in Bartholomew County.

presented in Figures 2 and 3 for two separate road locations with serious corrugation problems in Huntington and Jasper County, respectively. In both cases, the materials have coarser gradations compared to their county average although both gradations are on the finer side of the Indiana-specified gradation band. General characteristics of the corrugations measured are presented in Table 8. The characteristics include wavelength, or the distance between crests or troughs of corrugations; the depth from the top of the crest to the bottom of the trough; the CIV on the top of the crest and on the trough bottom; and a description of the location of the road in which corrugation occurred.

Wavelengths measured on corrugations varied from a low of 17 in (432 mm) to a high of 48 in (1220 mm), whereas depths varied from about 5/8 in (16 mm) to 1-1/4 in (32 mm) in different locations. From a review of past research studies on corrugations, Heath and Robinson reported a range of wavelengths between 12 in (300 mm) and 49 in (1250 mm) (*13*). The current study results also showed that shorter wavelengths may be associated with finer gravel materials and longer wavelengths with coarser materials. All material types are, however, susceptible to corrugation. Nevertheless, gravel and crushed rock with inadequate, nonplastic, or lost soil binder fines have been widely reported as susceptible to corrugation (*13-16*).

TABLE 8 SUMMARY OF AVERAGE CORRUGATION CHARACTERISTICS OF SELECTED ROAD SECTIONS IN THREE STUDY COUNTIES

County	Road Section	Clegg Impact Value			Wavelength (in.)	Depth (in.)	Location	
		Crest	Trough	Road			Description	Length (ft.)
Jasper	Div. 1	33	37	49	26	1.25	INT	150
	Div. 2	28	41	45	35	0.88	INT and SL	130
	400W	37	34	35	22	1.0	INT	150
	780W	52	36	54	29	0.8	INT	120
Huntington	300E 1	52	41	54	19	.63	INT and SL	180
	300E 2	50	45	50	18	0.88	SL	250
	100N	42	56	46	17	0.75	DW	150
Warrick	900W	47	49	60	32	0.75	SL	200
	475W	83	84	86	48	1.0	SLC	300

1. LOCATION DESCRIPTION
INT = Intersection ; SL = Slope ; SLC = Downslope before a curve
DW = Location near driveways

Because most surface materials found on Indiana county gravel roads were nonplastic and deficient in soil binder, corrugations are likely to be a continuing problem.

Locations and conditions on roads prone to corrugations include a downslope leading to an intersection and sloping road sections not near major intersections. In the latter case, the presence of a driveway accentuated the problem. This implies that gravel road locations in which vehicles are likely to decelerate and accelerate, usually to and from a stopped position, are likely to develop corrugations. The rate at which corrugations form, however, depends on the traffic volume. The 1-mi-grid local road network layout in Indiana, which requires frequent stops at intersections, provides the right conditions for formation of corrugations. Therefore, particular attention needs to be paid to maintenance of sections of road near intersections, on sloped sections, and sections with many driveways.

Previous studies have attributed the principal vehicle-related causes of corrugation to vehicle oscillation, and have suggested modifications to vehicle design to reduce this problem (*13*). The vehicle oscillation phenomenon could explain the differences in the CIV values on the crests and troughs of the corrugation waves measured in the current study. Both crests and troughs of corrugations in Table 8 exhibit equally high CIV values depending on which location is compacted the most by vehicle oscillation. The gravel or stone of uncompacted gravel material, such as that used by Indiana counties, is likely to be shoved and later compacted by decelerating or accelerating vehicles.

Recommended maintenance to improve corrugated gravel and stone road surfaces in the literature includes the application of granular materials with clay/silt binder, chemical treatment, watering, and more frequent maintenance consisting mainly of dragging or grading (*13*). Most Indiana counties increase the frequency of grading operations to overcome the problem of corrugations. However, based on this study, a review of county material use practices would be beneficial. An increase in the binder content in the surface gravel material could improve the general performance of gravel roads, especially near intersections and on slopes. Stones above 1 in. in size that may inhibit effective traffic compaction of the surface gravel and increase roughness should be avoided.

CONCLUSIONS

The results of the analysis presented in this paper show that material gradation properties represented by the percentage passing ASTM No. 10 and No. 40 sieves, sieve size passing 95 percent of the material, fineness modulus, and liquid limit are all significantly correlated with both roughness and average rut depth. However, average rut depth, road surface strength measured by the CIV, and the sloping characteristics of the road were also found to have significant correlation with roughness. There was a significant difference between the strength of the gravel surface courses in the counties studied. Average road width and the percentage of material passing a No. 200 sieve were also important variables.

Although counties usually apply surface gravel without rolling, the research results showed that well-compacted roads were less rough. If lowering roughness is considered important by local authorities, then it may be justified to roll the surface gravel immediately after application and before it significantly varies from an optimum moisture content. The higher density should facilitate the formation of a surface crust during drier weather. Such surface materials should contain an adequate soil binder to perform well.

Material properties provide some explanation for the formation of corrugations and ruts, and measured gravel and stone road surface condition such as roughness, average rut depth, and strength. Blading or grading can reduce roughness by over 50 percent. However, the frequency of grading should be increased on gravel roads at sloping locations with many driveways or sections near major intersections in which corrugations are likely to occur.

Although test sections of 1 mi (1.6 km) were used in this study, the results varied significantly within a section. Additional data would improve the accuracy of the results.

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Automated Data Acquisition for Low-Volume Road Inventory and Management

THEODORE HOPWOOD II, GARY W. SHARPE, JOHN W. HUTCHINSON, AND ROBERT C. DEEN

Local governments need suitable inventories and condition surveys to accurately determine and rank their road funding needs. An efficient method of performing an inventory of the roads and determining the condition of major system components is to use an automated data acquisition system to measure and record essential data. The data can be collected and processed by a central inventory management group. The inventory should include pavement characteristics, roadway geometrics, and roadside features. Pavement condition can include structural integrity, road roughness, and skid resistance. Roadway geometrics include horizontal curvature, length, width, superelevation, and grade. Roadside features include signing, intersections, guardrails, and obstacles. A concept is presented for a second-generation vehicle-mounted photologging system that can, in a single pass, photolog the roadway and automatically record measurements necessary to inventory and rate roadways. The major functions of a central inventory management group are also described. Second-generation photologging systems use either camera or video systems to record visual data. Particulars of those methods are presented and compared. Sensors for additional data measurement, including grade, superelevation, and road roughness, are described. Data recording methods are discussed with emphasis on data storage technological improvements. The function of the inventory management group is to obtain data and convert it to a usable form. This consists of five basic activities: operations management, data acquisition, data reduction, data interpretation, and inventory preparation. Data reduction includes sorting and editing visual records (film or videotape) and digitizing analog recordings. Several methods for addressing those records and cross-correlating visual and sensor data are discussed. They range from manual sorting techniques to computerized, laser-disc data-processing systems. Data interpretation requires a review of acquired data in its formatted, addressable form. Analyses can include a combination of visual and sensor analyses for pavement condition and roadway geometrics. Other geometric-related analyses can be based almost entirely on visual recordings of roadside features. In some instances, such as unpaved roads, in which few accepted standards exist, new standards or criteria should be formulated for evaluation. Otherwise, accepted standards should be used. The road rating process and methods employed to rank pavement and traffic safety-related rehabilitations are discussed. The interaction between the central inventory management group and the low-volume road agency in preparing the final compiled inventory is also discussed.

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The management of local roads can be enhanced by use of an inventory of the physical elements of the roadways and a survey of their existing conditions. Such inventories are commonly used as tools to (a) determine future funding requirements, (b) assess current funding needs, (c) allocate and manage limited funds, (d) rank construction and repairs, (e) increase safety, and (f) avoid unnecessary accident-related litigation.

In order to obtain valid inventories, local agencies must send people to the field either to obtain information about the infrastructure or to verify data obtained from office files and records. Field work is an absolute requirement to obtain condition surveys. Field and office data must be condensed into a usable form and evaluated. The results must be reformatted for presentation to nontechnical users.

Local agencies often encounter problems when performing inventories. It is difficult to locate and train competent survey personnel. The cost of manual data acquisition is expensive. Processing field data may also prove to be costly and troublesome. It may be difficult at the local level to obtain technical expertise to evaluate and analyze the data. Finally, local agency personnel may not be sufficiently skilled or experienced to render the inventoried data and ratings in a manner that can be understood by laypersons such as county judges, law enforcement personnel, and other local government officials and employees.

One method of overcoming those difficulties is to employ automated drive-over data acquisition. Data acquisition is performed by a crew in a van or car equipped with photologging or videologging systems, often supplemented with other nonvisual data acquisition systems that measure a variety of pavement parameters (Figure 1). The derived data are automatically stored in an addressable form so that they can be

FIGURE 1 Modern photologging vehicle for drive-over roadway surveys. *(Photograph courtesy of Highway Products International, Paris, Ontario, Canada)*

retrieved later. When the field data have been obtained, it can be delivered to a central inventory management group. That organization can economically and properly format and analyze the data and produce usable inventories and synopsized reports.

A number of companies manufacture assembled data acquisition systems for pavement surveying and market fully equipped vehicles. Several consulting firms nationwide offer or are preparing to offer pavement survey and inventory services.

Local agencies that have large road inventories of 2,000 or more miles might investigate purchasing an equipped data acquisition vehicle. Road agencies that have smaller inventories might consider employing a pavement survey and inventory consultant.

A third approach is for local agencies to pool resources and contract with a university-associated transportation program, a technology transfer organization, or a consultant to establish a central inventory management group. Either organization would be likely to have access to the varied and specialized technical skills necessary to establish a competent roadway management team.

The road inventory should include a list of the physical elements and their locations, pavement and road surface conditions, and roadway geometrics. Pavement condition should include pavement type, pavement structural integrity, road roughness, and skid resistance. Roadway geometrics should include roadway length, pavement width, superelevation, grade, and roadside geometrics such as signing, intersections, guardrails, and roadside obstacles.

PHOTOLOGGING

The primary form of field data acquisition is photologging or videologging. Acquisition of roadway visual images is termed first-generation photologging and has been performed by state highway agencies for about 20 years (*1*). Considerable information related to pavement and general roadway environment can be acquired from photographs of the in-place road system. This process, augmented by concurrent acquisition of nonvisual data, is termed second-generation photologging. It provides for economical acquisition of data parameters over the entire route. As the visual image and nonvisual parameters are being recorded concurrently, often on the same data storage medium, it is easy for central inventory management personnel to correlate individual visual records with attendant nonvisual data. This allows correlations to be made that would not be possible if the visual image inventory was conducted separately from a statistical sampling of the nonvisual data. It should be noted that 100-percent data acquisition by photologging is difficult to achieve and that a limited amount of more conventional field inspections should be anticipated. Forty-three states currently perform some type of photologging or videologging operation.

Visual Data

Commercially available photologging or videologging systems employ one or more vehicle-mounted cameras aimed forward, rearward, or downward, depending on the manufacturer. If more roadside detail is sought, as would be the case for low-volume roads, the camera(s) could be aimed slightly outward. If four-lane roads are being recorded, it is possible to film or tape the two lanes in a given direction simultaneously with one camera. If only a visual inventory is being performed, it is also possible to film or tape a two-lane road in one drive-over pass. A front-mounted camera could record one lane and a rear-mounted camera could record the other. When very general visual records are desired, a single pass with one camera may suffice. When second-generation photologging is conducted, one pass may be required for each lane, which would limit the utility of additional cameras.

One use of visual data is to determine distances. As shown in Figure 2, the height of a telephone pole can be determined by measuring the image height in successive film frames. The height of the pole can then be computed from the following equation:

$$D = d_1 d_2 S \cos \phi / (d_1 - d_2) f R$$

where

D = dimension of the object (m),
d_1 = size of the image on frame one (mm),
d_2 = size of the image on frame two (mm),
S = frame interval (m),
ϕ = angle of skew of the camera to the roadway,
f = focal length of the lens (mm), and
R = enlargement ratio, which is equal to the ratio of the size of the image on screen to the size of the image on film.

Distances to the front and side of the image can be roughly measured using a perspective overlay (Figure 3). The road should be level if a front-viewing camera is used. Distances on hilly roads can be measured using the perspective overlay with a rear-directed camera.

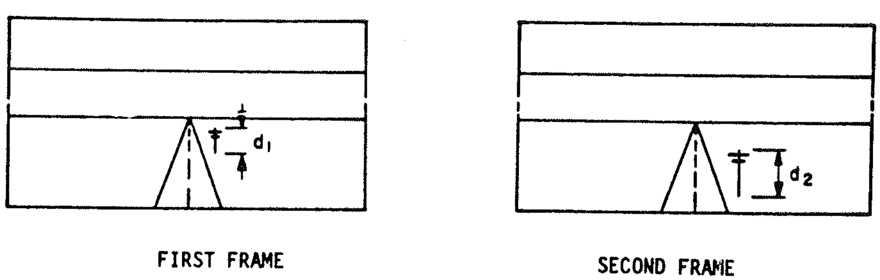

FIGURE 2 Increase in size of visual image (telephone pole) with successive picture frames. *(Illustration courtesy of TECHWEST Co., Richmond, British Columbia, Canada)*

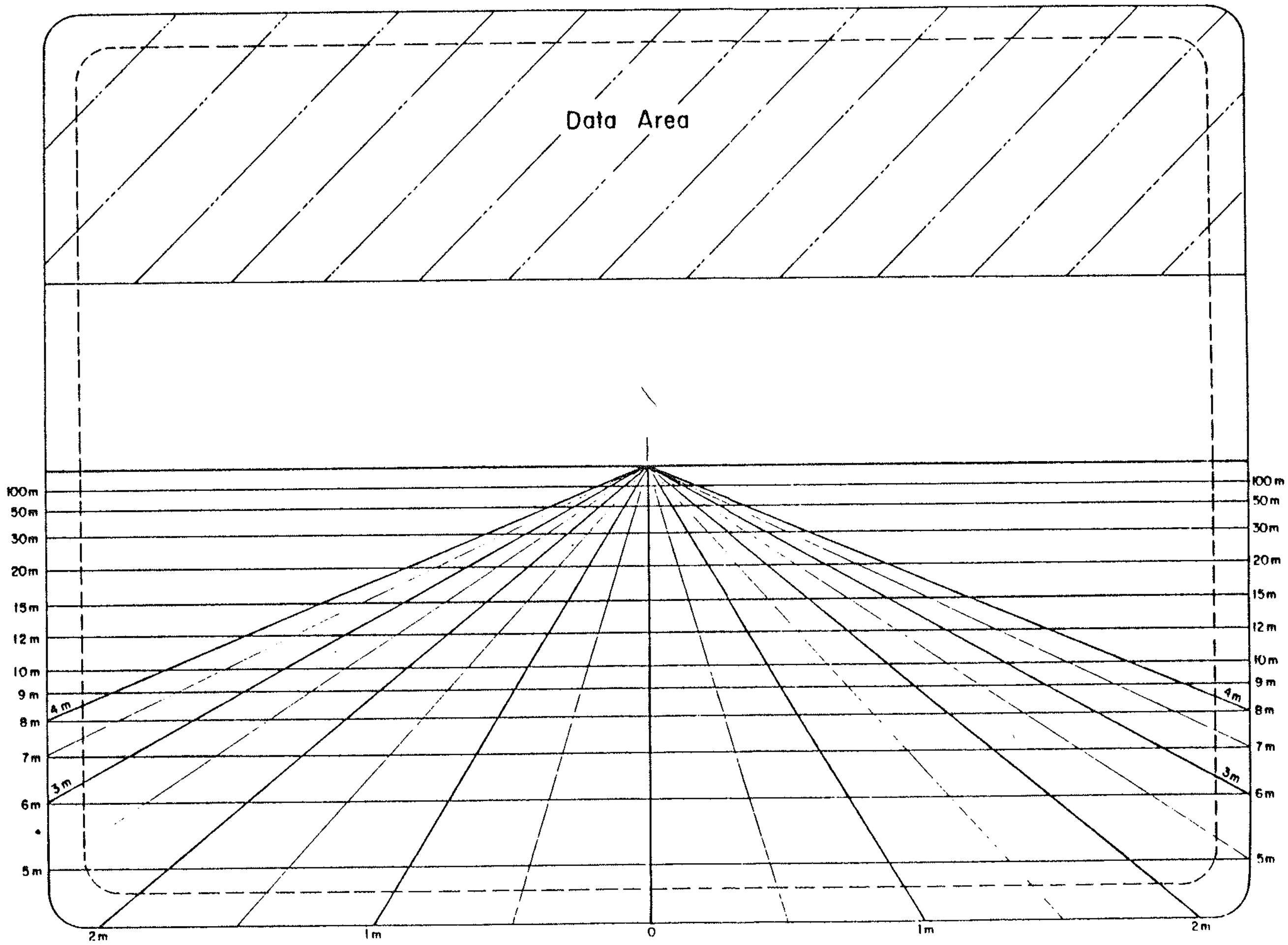

FIGURE 3 Perspective overlay for deriving dimensional data from photologging images. *(Courtesy of TECHWEST Co., Richmond, British Columbia, Canada)*

Color film or videotape is usually used. There are advantages and disadvantages to both film and videotape recording methods. Camera film must be developed whereas videotape is immediately ready for replay. It is cheaper to copy videotape and it can be erased and reused when necessary. In addition, several video images can be superimposed and displayed simultaneously as split-images on a video monitor. Camera film can show finer detail than videotape. Film could be necessary when small pavement flaws such as tight cracks are to be detected visually. Video images are usually taken continuously and are best reviewed at real-time recording speeds. However, motion picture film can be exposed at fixed intervals along the road (one frame approximately every 50 ft). When that film is replayed for data review and interpretation, central inventory management personnel will benefit from the derived time-compression gained during playback.

Even when first-generation photologging or videologging is employed, it is desirable to log information such as the logging date, route, agency control number, odometer reading, vehicle speed, tape or film number, and operator comments. System manufacturers provide for the visual superimposition of information onto a portion of the film or video image (Figure 4). If second-generation photologging or videologging is performed, nonvisual sensor information can be digitized, encoded, and stored together with the conventional data on a portion of the visual image. If visual data storage is not desired, the soundtrack of either the film or videotape can be used to store analog or digitized data.

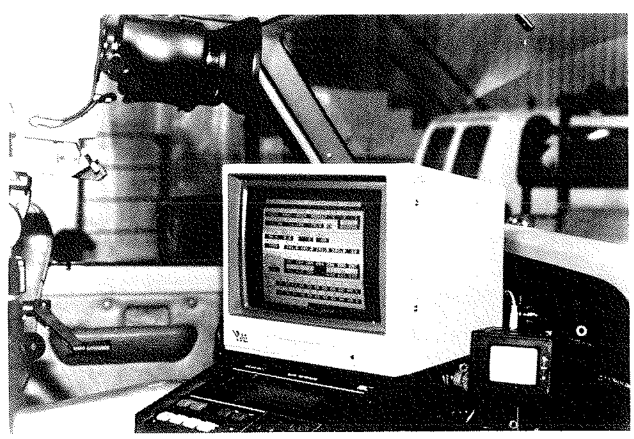

FIGURE 4 Visual display of coded test information in videologging vehicle. *(Courtesy of Highway Products International, Paris, Ontario, Canada)*

Nonvisual Data

Second-generation data acquisition systems are capable of measuring and recording information that cannot be determined visually, such as road roughness, pavement surface side friction, grade, cross-slope, bearing, and horizontal and vertical curvature.

Road roughness can be measured by the deflection of the rear axle of the data acquisition vehicle. This can readily be measured by attaching an accelerometer to the axle for small displacements. One manufacturer supplies a separate, towed road roughness measuring system capable of measuring roughness wavelengths from 1 in to 300 ft (Figure 5). The road roughness system can be calibrated to national standards to compare data with that of other road agencies.

Side friction of the pavement surface can be measured by use of a ball-bank indicator in which the angle of swing of a damped pendulum is expressed as the tangent of the angle. This is a measure of the sideways frictional forces between the tires and the road surface as the vehicle travels along a curve. When correlated with vehicle speed, this measurement ascertains whether the superelevation of the curve is safe for its posted speed.

Grade measurements are usually made in reference to a gyroscope. Readings are given in percent of grade with a plus or minus sign to indicate slope. Cross-slopes or transverse-slopes are measured in a similar fashion. A gyrocompass is used in one manufacturer's system to determine bearing. A combination of bearing and odometer signals provides for determination of horizontal curvature. Grade (gyro) signals also can be combined with odometer signals to determine vertical curvature.

Some first-generation photologging or videologging systems can be built up to second-generation systems as funding allows. All photologging, videologging, or nonvisual data acquisition systems should be broken into components to the maximum extent possible to provide for replacements when better technology becomes available.

A third generation of pavement data acquisition vehicles is currently being evolved. Those systems will contain instrumentation to make impromptu analyses of pavements that currently do not lend themselves well to conventional data acquisition or follow-up analysis. The vehicle shown in Figure 1 is equipped with sensors in an enlarged front bumper that detect rutting. Swedish and Japanese firms have developed laser sensors that are capable of detecting a number of types of distress in pavements, including cracking and rutting (*2*). An American firm is currently developing a fully automated system that will optically survey pavements and assess their condition.

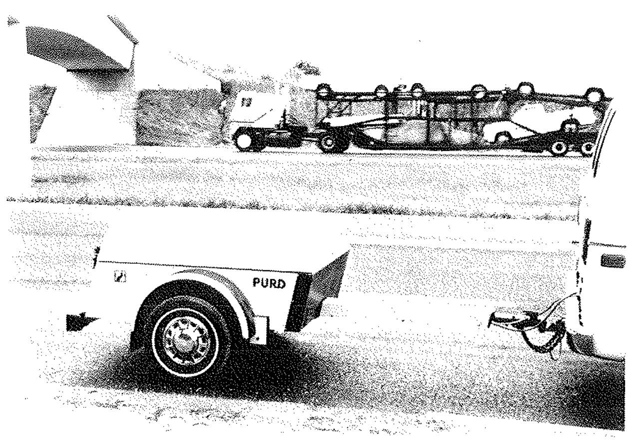

FIGURE 5 Towed road roughness measurement system. (*Courtesy of Highway Products International, Paris, Ontario, Canada*)

Structural evaluations are generally applied to pavement sections on the basis of perceived needs as determined from visual inspections of pavement distresses and the observation of load- and fatigue-related distresses. There are a number of devices and procedures available to collect and process deflection data for structural evaluations of pavements. Such equipment does not lend itself well to impromptu data acquisition methods. It is hoped that more rapid test methods will be developed that potentially could be "piggy-backed" onto the second- or third-generation photologging vehicles.

OPERATIONAL REQUIREMENTS FOR DATA ACQUISITION

Vans are the ideal test vehicles for data acquisition. They have sufficient internal space and can be serviced at most locations should the need arise. The test van should have space to house the cameras, data entry devices, the photologging or videologging control panel, electrical generating equipment, and spare film or tape. A crew of two, a driver and a system operator, is generally required. The system operator logs route information, tape numbers, and special events on the manual data entry system. He changes the film, checks how well the system is functioning during the tests, and directs the driver.

The vehicle should generally be operated at speeds between 35 and 55 mph. Depending on the travel time to the test site, the data acquisition crew could record 500 to 1,300 lane-miles of road per week. The lower figure is representative of city operations. The higher figure is possible on open high-volume paved roads. The Connecticut Department of Transportation employs a photologging system. In order to complete the inventory of the 4,000 mi of pavement in Connecticut, 8 months of second-generation photologging is accomplished each year. This had previously been done manually by six two-man field crews in an 11-year cycle.

For low-volume urban roads, 100 ft of film is taken every 25 mi with an exposure interval of one frame every 35 feet. A week of city photologging could take up to 20 reels of film. If a video system is used, 20 2-hr tapes are required for the same operation. A two-man crew could conceivably photolog about 50,000 lane-miles each year with an 80-percent duty cycle. In rural locations, the exposure interval could be increased to one frame every 70 feet, which would increase the total amount of miles covered on a can of film or a videotape.

Costs for different types of inventory operations, including data reduction, vary according to the category of road inspected and the number of data parameters to be inventoried. Costs per mile typically range from $134 for urban areas with high sign densities to $15 for rural areas with very low sign densities (*3*). Photologging and videologging have corresponding costs that range from $67 and $69, respectively, for urban areas to $23 and $16, respectively, for rural areas. However, these costs represent only one type of inventory: signing. If multiple data parameters are required, the photologging and videologging methods are even more economical.

INVENTORY MANAGEMENT GROUP

A central inventory management group normally consists of five teams: the operations/management team, the data acquisition team, the data reduction team, the technical review and rating team, and the inventory preparation team (Figure 6).

The operations/management team supervises and coordinates the entire central inventory management group. They interface with other units or agencies responsible for various aspects of the low-volume road system and determine test requirements. The operations/management team schedules work and coordinates all operations. They also review all compiled inventories and synopses before they are delivered to other units and staff.

Once the data acquisition team has obtained data from the low-volume road system, the raw data are transmitted to the data reduction team. That team consists of several technicians who develop film and digitize and computerize analog sensor data. They inspect photologged or videologged images to see if they are suitable for analysis. The technicians also check to see if all sensor data are reasonable and review the routing and identification information to ensure that data are complete. Finally, they organize the data into a reduced, formatted form to facilitate ratings and analyses by the technical review and rating team.

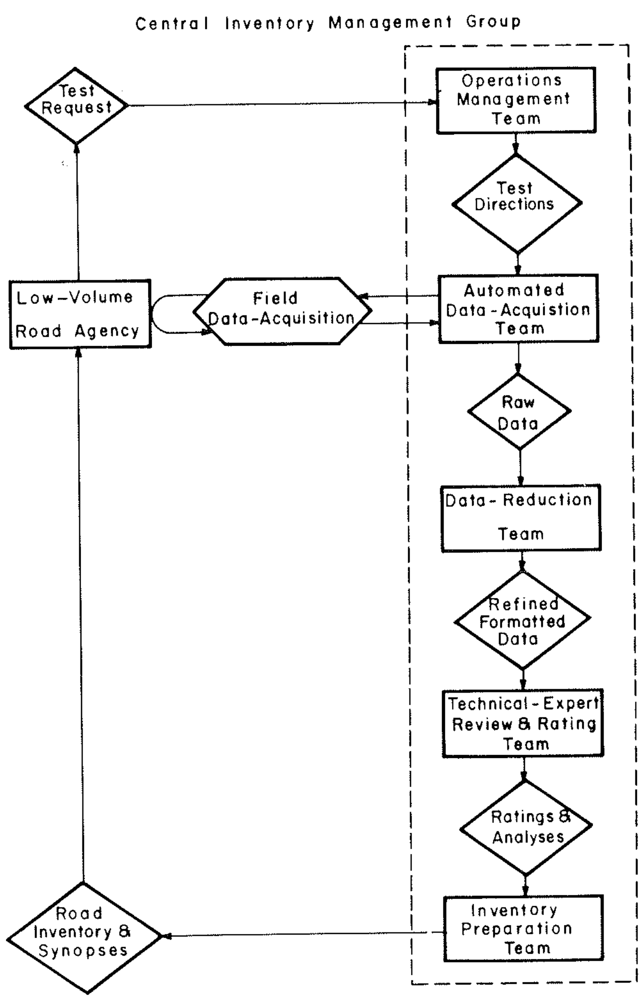

FIGURE 6 Organizational chart for a central inventory management group.

The technical review and rating team normally consists of a few engineers and technicians with expertise in the fields of pavement analysis and rating, pavement management, and transportation engineering. They review the reduced data and correlate those findings with visual records obtained by photologging or videologging. They rate pavements, identify deficient areas, determine remedial measures, rank repairs, determine repair costs, offer alternative solutions to field problems, and provide other technical assistance that would be helpful to the low-volume road unit.

Pavement ratings, roadway analyses, and other data are then forwarded to the inventory/preparation team. They organize a compiled inventory of the road system that includes the required reports or synopses that explain in lay terms the required funding levels and suggested priorities for repair of deficiencies identified in the system. Once the final reports and inventory are completed, the inventory, synopsis, and film or tapes are sent to the operating unit for their use.

POST-INSPECTION DATA PROCESSING

Nonvisual Data

Because of the amount of information that must be assembled, retrieved, and analyzed, it would be cost-effective to digitize as much nonvisual quantitative data as possible and then process and correlate those data by computer. Data should be stored in the field in analog or digital form on the soundtrack of the film or videotape. That data can then be easily off-loaded from the completed film or tape, digitally formatted, and computer-processed. Pattern recognition computer programs should be developed to detect and identify problem areas in the pavement. The entire data set could be automatically scanned and evaluated on a preliminary basis with a low expenditure of manpower. Computer data base management programs should be used to store and manipulate data sets. The final evaluations should be prepared using computer spreadsheet programs that can easily be incorporated into completed inventories and road system synopses.

Visual Data

If large data bases are accumulated, the many reels of film or videotape cassettes could be difficult to catalog and store. If the low-volume road agency desires to continually inspect or review the visual records, a more convenient means of storing and processing data should be adopted. The Connecticut Department of Transportation currently employs such a system (*4, 5*). In the course of photologging 4,000 mi of highways, 660 100-ft reels, or 920,000 frames of 35-mm film, are used. The film is sent to a professional processor and converted to laser videodiscs. During the conversion process, the image in each frame is enhanced and improper lighting is corrected. The laser videodiscs store four gigabytes of data on a laser disc the size of a 12-inch phonograph record. Each videodisc stores about 110,000 frames of data.

The image conversion process costs approximately $10,000 per videodisc; additional copies cost about $20 each. This provides for inexpensive backup of data. Laser discs are also more durable than either film or videotape. The costs of image conversion can be recaptured through reduced expenditures for files, storage space, and time expended in locating the required data. Videodiscs are also formatted and combined with a data retrieval system that allows 3-second access to any location stored on the disc.

The visual data can be analyzed by projecting the cinematic film or videotape and reviewing the images. The exposition of nonvisual test parameters on a portion of the screen would greatly aid in concurrent evaluation of data (Figure 7). It would also be useful to have a script of the preprocessed computer data to aid in rapid identification of problem locations. When technical experts review the film or tape, they could be provided with digital encoding devices to record features and visually rate the pavement roadway and roadside. Such records could ideally be incorporated with an adaptive-learning or artificial-intelligence computer program to reveal visual and quantitative relationships not readily discernible, even to technical experts. This process would eventually yield a higher order of data analysis, and would allow engineers and technicians to perform some visual analyses, thereby freeing the technical expert to seek other significant parameter relationships. At least one data acquisition firm offers laboratory equipment to perform some of these processes.

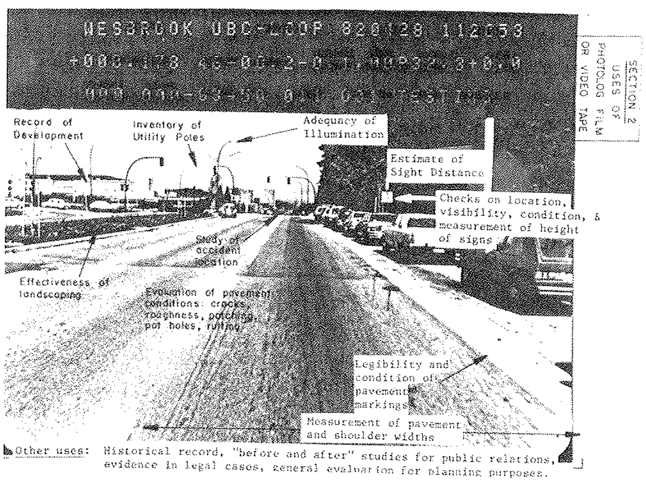

FIGURE 7 Visually detectable information provided by photologging. (*Photograph courtesy of TECHWEST Co., Richmond, British Columbia, Canada*)

INTERPRETATION OF DATA

Interpretation of derived data by the technical review team requires input of specific types of visual, nonvisual, and qualitative and quantifiable pavement data. The technical review is broadly divided into two categories: pavement and road surface evaluation and management, and traffic and roadside environment evaluation and management. Both reviews often need the same nonvisual data parameters and visual images.

One necessary component required for either review is complete geometric identification of each road; functional classification of the route; identification of section termini and block or section number; length of section; pavement type; pavement width; number of travel lanes; and type and width of shoulders or curbs. Other additional data not provided by the drive-over tests include traffic data such as average daily traffic (ADT) and vehicle compositions; accident data such as frequency of occurrence, severity, and exact locations; and planning information such as impending zoning changes and land utilization.

PAVEMENT AND ROADWAY DATA

In order to rate pavement and roadway surface condition, it is necessary to identify the current problems on the roadway that require correction. Condition ratings can be compared with prior ratings to determine deterioration rates and to judge the effectiveness of rehabilitation strategies. For example, condition ratings can be used to develop deterioration versus time or traffic volume (fatigue) curves.

Pavement condition ratings can be determined on the basis of a number of factors or combinations thereof. The following are factors involved in condition ratings for pavements:

- Ride quality,
- Observable distress,
- Findings from structural evaluations,
- Other factors such as accident records or skid test results, and
- A combination of the above, or other, factors.

Observable distresses vary with the pavement type. For bituminous-surfaced roads, these typically include alligator cracking, block cracking, reflection cracking, rutting, raveling, bleeding, shoving, corrugations, potholes, and patching (Figure 8). Observable distresses in rigid pavements include blowups, corner breaking, "D" cracking, faulting, spalling, transverse cracking, longitudinal cracking, popouts, pumping, polished aggregate, joint and seal deterioration, and patching. Observable distresses associated with aggregate-surfaced roads include rutting, corrugations, potholes, aggregate loss, slipperiness, surface erosion, and dusting.

The manifestation of distress of any pavement or road surface is an indication of defects in materials or the overall structural integrity of the road. The photologging survey is an important part of a pavement distress survey.

Once the roadway condition is established and rates of deterioration are predicted, repair strategies can be formulated.

FIGURE 8 Bituminous pavement showing alligator cracking.

Those strategies can be selected with knowledge of the identified deficiencies, wear rates, and maintenance funding available. The pavement management staff can offer a number of repair strategy "menus" from which the local agency can select. Various strategies could be prepared to show what work can be performed at various levels of funding. This is also necessary because it is difficult to immediately determine local needs. However, all pavement areas that constitute a motoring hazard would need to be identified and recommended for prompt repair.

TRAFFIC AND ROADSIDE ENVIRONMENT DATA

The traffic engineers can analyze road geometry obtained from visual image analysis, traffic volume and accident data, and some nonvisual data obtained during the drive-over inspection. Traffic engineers can visually determine if the traffic volume warranted a higher standard of surface maintenance, traffic control (signing, marking, and delineation), and geometrics (stopping sight distance and focusing sight distance) than is currently existing or planned. Traffic engineers could also note local land use, planning, and zoning. They can provide recommendations for future traffic routing and updating of existing facilities.

The visual and visually derived data can be used in conjunction with accident data to identify causal relationships and derive remedial changes in the roadway or signing. Obvious road hazards such as narrow steep shoulders could easily be detected. Significant hazardous roadside features or illegal access roads can also be identified for follow-up action. Poorly placed, missing, damaged, or illegible signing could be identified for prompt replacement. Visual data can also be used to identify locations in which traffic codes are enforced, but in which local conditions necessitate change. Numerous braking skid marks at an intersection are a good indicator of the need to revise a particular roadway element.

Nonvisual data such as crown, grade, or superelevation, correlated with visual images to detect signing and sight distance, can be used to identify dangerous or substandard locations. In many cases, legal speed limits may exceed safe speeds. A reduction in the speed limit would minimize such hazards.

The traffic engineer can recommend solutions for every substandard or hazardous location. He can also assign a risk factor to each location based on the severity of the defects. He can determine the probability of accidents at the locations based on the traffic volumes, and predict the probable consequences of accidents.

Information on the traffic and roadside environment can be compiled along with a list of repair strategies to upgrade the signing and make geometric modifications to correct deficiencies. Areas that require immediate action can be identified. Alternative strategies can be prepared, showing safety improvements that might be expected, in terms of accident or risk reduction, for various levels of funding directed toward different tasks. For a given level of funding, strategies necessary to achieve the maximum improvement in safety can be identified.

Implementation of a rational safety-related roadway management program would render agencies less susceptible to accident-related litigation. Failure to perform safety maintenance as a result of ignorance of standards is not a legal defense. When safety-related work was not performed because of limited funding or because the direction of available funds to other items was deemed more critical, it could be argued that the agency was operating responsibly. Not every safety-related problem detected through the inventory process needs to be repaired immediately, especially if risks are assessed.

Agencies should balance expenditures among local needs with available funds. The technical review and rating team should provide recommendations for allocating the funds between the pavement and traffic sectors.

CONCLUSION

It should be noted that, for appropriation purposes, only a statistical sampling of a local road system needs to be inventoried and analyzed. A complete inspection and analysis is necessary to assist in ranking remedial work. The primary purpose of automated data acquisition and centralized inventory management is to determine desirable funding levels. A second purpose is to aid in directing available maintenance funds to the most severe and beneficial strategies. Once the inventory and condition survey system is in place, it can be used for a variety of additional purposes, including a cost evaluation for the purchase of special equipment, a plan for upgrading unpaved roads, an evaluation of the long-term performance of specific repairs or designs, and an evaluation of roadside drainage conditions. The system can also be used to plan communities and development, monitor utilities, recruit new businesses, zone properties and districts, and even provide a historical record of the community. If it is properly planned and implemented, this operation could provide a number of unanticipated uses to the low-volume road agency and the community it serves.

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A Computer Model for Developing Road Management Strategies

SARA E. BALDWIN, MARTIN J. HANSON, AND MICHAEL A. THOMPSON

The development of road systems in formerly unroaded agricultural areas provides an opportunity to apply road spacing theory. This analysis focuses on minimizing the cost of harvesting and transporting timber. A model is developed to optimize road spacing for three standards of road in regions with flat terrain and a uniform crop distribution. Road construction, harvesting, and haul costs are variables in the model. Optimum, economic road spacings can be determined from the model and used as a guide in developing a harvest plan. The effect of the soil condition and harvesting method on optimum road spacing is demonstrated through examples.

Underdeveloped road systems in forested areas with gentle terrain provide an opportunity to apply road spacing theory (*1*). This theory was first applied to logging operations in 1942 by D. M. Matthews (*2*). Matthews formulated a mathematical model to determine how roads could be spaced to minimize the combined costs of harvesting and transportation.

Harvesting costs relevant to the problem are the costs of skidder operation and maintenance as a function of skid distance. Transport costs include road construction and truck operation and maintenance.

Matthew's theory is discussed on the basis of total relevant cost per unit volume of wood produced. One standard of road is used to analyze the influence of soil condition and skidding equipment on optimum road spacing. An example of an application that uses typical costs from the Great Lake States region (Minnesota, Michigan, Wisconsin, Illinois, Indiana, and Ohio) is shown.

The basic equation is then expanded to include the effect of haul costs and different road standards. The influence of different logging trucks, skidding equipment, and soil condition is analyzed for the expanded model. A microcomputer program of the model is used to examine the effects of soil condition, desired road standard, equipment type, and harvest volume on economic road spacing. This information can then be applied to harvest planning and used to minimize costs.

THE BASIC MODEL

Matthew's model optimizes road spacing by balancing the costs of skidding and road construction (Figure 1). This model is based on the following assumptions:

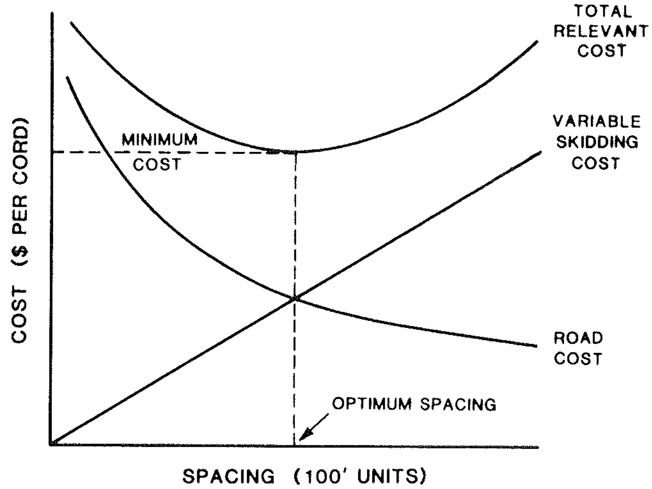

FIGURE 1 Optimizing road spacing by Matthews' approach.

- The harvest area is on flat terrain.
- Logs are skidded to the roadside from both sides of the road, where they are loaded onto logging trucks.
- The timber is distributed uniformly over the harvest area.
- Only one standard of road is used for access.
- Roads are laid out parallel and evenly spaced.

The physical layout of this problem is shown in Figure 2. This application of Matthew's theory compares costs on a unit volume basis. In the development of the basic model, a road cost and skidding cost per unit volume form the total relevant cost equation.

First, the road cost portion of the total relevant cost equation is developed. The following variables are used to develop this part of the optimum road spacing model:

R = road cost per station (station = 100 feet),
S = road spacing in stations, and
V = volume of timber per acre (1 acre = 43,560 ft^2).

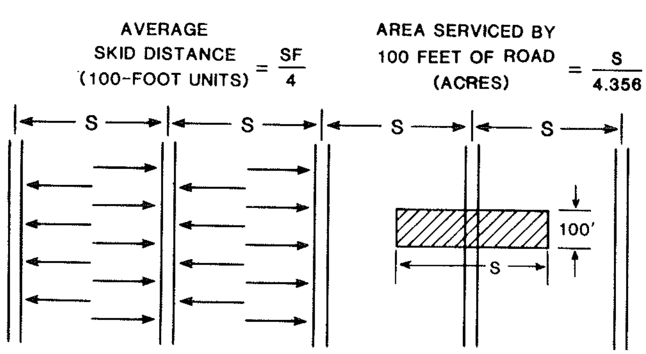

FIGURE 2 Road layout and unit area for the basic model.

S. E. Baldwin, USDA Forest Service, 310 W. Wisconsin Ave., Suite 600, Milwaukee, Wis. 53203. M. J. Hanson, Barrientos and Associates, Inc., 3822 Mineral Point Rd., Madison, Wis. 53701. M. A. Thompson, USDA Forest Service, Forest Sciences Laboratory, Forest Hill Rd., Houghton, Mich. 49931.

Road cost per unit area is used to develop a road cost per unit volume. The theoretical unit area is shown in Figure 2.

Road cost per unit area = $4.356R/S$

The unit area is defined in acres. The factor of 4.356 is derived as follows:

$[R(\$/100 \text{ ft}) \times 1/S(\text{stations})$
$\times 1/100(\text{ft/station})]/43{,}560 \text{ ft}^2/\text{acre}$
$= 43{,}560 \times R / 10{,}000 \times S \times \$/\text{acre} = 4.356R/S$

Dividing the road cost per unit area by the volume per unit area simply changes the relationship to the following:

Road cost per unit volume = $4.356R/SV$

The skidding cost portion of the total relevant cost equation is then determined. The additional variables required are as follows:

C = machine rate or hourly cost of skidding (\$/scheduled hour);
T = distance-dependent skidding time, which is the inverse of machine speed (min/station);
F = sinuosity factor (proportion of actual skid distance to straight line distance);
U = average load size of skidding equipment (same volume units as V); and
A = actual average skid distance (Figure 2)
= sinuosity $\times$ spacing/4
= $FS/4$

Skidding cost per unit volume
= $[2 \times T(\text{min/station}) \times A \text{ (station)} \times C$ (\$/scheduled hour)(scheduled hour/60 min)]/ U (volume units)
= $(2 \times F \times S \times T \times C)/(4 \times U \times 60)$
= $TSFC/120U$

The total relevant cost model is formed by combining the road and skidding costs. The cost relationships are shown in Figure 1.

Total relevant cost per unit volume = $4.356R/SV + TSFC/120U$

The purpose of this analysis is to develop a relationship to calculate the road spacing that results in a minimum cost. Applying the maxima-minima theory of calculus to this equation, the derivative with respect to spacing is set equal to zero:

$d(\text{total cost})/dS = 0$
$-(4.356 \times R)/(S^2 \times V) + (T \times F \times C)/(120 \times U) = 0$

This suggests that the optimum road spacing is as follows:

$S_{\text{opt}} = (522.72RU/TFCV)^{1/2}$

Recent work has been performed on the effect of overhead costs on road spacing. Thompson analyzed this relationship and found that by assuming the harvesting system is in balance (i.e., skidding capabilities do not exceed the felling and limbing capabilities), the system productivity will be the same as the skidding productivity (*3*). Costs independent of production (i.e., overhead costs) will vary with the system productivity and with road spacing (Figure 3). Examples of overhead costs are administration, bookkeeping, supervision, procurement, shop facilities, maintenance equipment, fuel equipment, machine transport equipment, and inventory. Because productivity is directly related to road spacing, overhead is a necessary factor to consider in the model. When overhead costs are added to the model, the equation becomes:

Total cost per unit volume = $4.356R/SV + TSFC/120U + TSFQ/120U$

where

Q = overhead costs (\$-scheduled hour)

Optimum road spacing for the basic model then becomes:

$S_{\text{opt}} = [522.72RU/TFV(C + Q)]^{1/2}$

The variables R, U, T, F, V, C, and Q will be known for a given logging situation. Optimum spacing can be determined by applying the estimated conditions to the model.

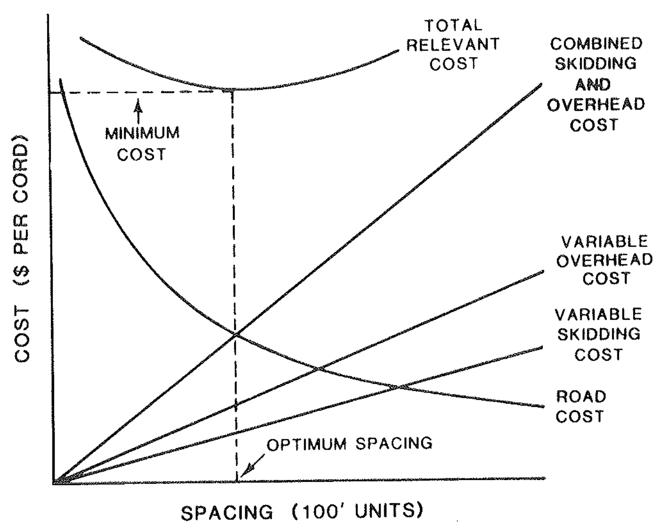

FIGURE 3 Optimizing road spacing by including overhead costs.

AN EXAMPLE

This mathematical model was developed for the harvest situation that utilizes one road standard on flat terrain. The application of this model is demonstrated by using three common skidding methods in three typical soil conditions. Data for these examples are taken from road construction and skidding operations in the Lake States region (Figure 4). Each cell contains the spacing versus volume relationship for a particular combination of skidding method and soil condition. These relationships are represented as graphs in Figures 5, 6, and 7. A particular timber volume removed per acre has a corresponding optimum road spacing.

$$\text{Opt. } S = \sqrt{\frac{522.72RU}{TFV(C+Q)}}$$

SKID METHOD \ SOIL TYPE	DRY	WET	ROCKY
80 Hp Cable Skidder	R = \$300/Sta U = 0.5 Mbf T = .35 Min/Sta F = 1.15 C = \$35/SH Q = \$35/SH $S = \sqrt{\frac{2783}{V}}$	R = \$435/Sta U = 0.5 Mbf T = 0.6 Min/Sta F = 1.25 C = 36.75/SH Q = \$35/SH $S = \sqrt{\frac{2113}{V}}$	R = \$400/Sta U = 0.5 Mbf T = .45 Min/Sta F = 1.20 C = \$36.75/SH Q = \$35/SH $S = \sqrt{\frac{2698}{V}}$
60 Hp Forwarder	R = \$300/Sta U = 1.1 Mbf T = 1.0 Min/Sta F = 1.2 C = \$30/SH Q = \$35/SH $S = \sqrt{\frac{2212}{V}}$	R = \$435/Sta U = 1.1 Mbf T = 1.25 Min/Sta F = 1.3 C = \$31.50/SH Q = \$35/SH $S = \sqrt{\frac{2315}{V}}$	R = \$400/Sta U = 1.1 Mbf T = 1.0 Min/Sta F = 1.25 C = 31.50/SH Q = \$35/SH $S = \sqrt{\frac{2767}{V}}$
110 Hp Grapple Skidder	R = \$300/Sta U = 0.95 Mbf T = 0.3 Min/Sta F = 1.10 C = \$50/SH Q = \$50/SH $S = \sqrt{\frac{4514}{V}}$	R = \$435/Sta U = 0.95 Mbf T = 0.55 Min/Sta F = 1.20 C = \$52.50/SH Q = \$50/SH $S = \sqrt{\frac{3193}{V}}$	R = \$400/Sta U = 0.95 Mbf T = 0.4 Min/Sta F = 1.15 C = 52.50/SH Q = \$50/SH $S = \sqrt{\frac{4213}{V}}$

FIGURE 4 Optimum road spacing calculations for three skidding methods in three soil conditions (basic model).

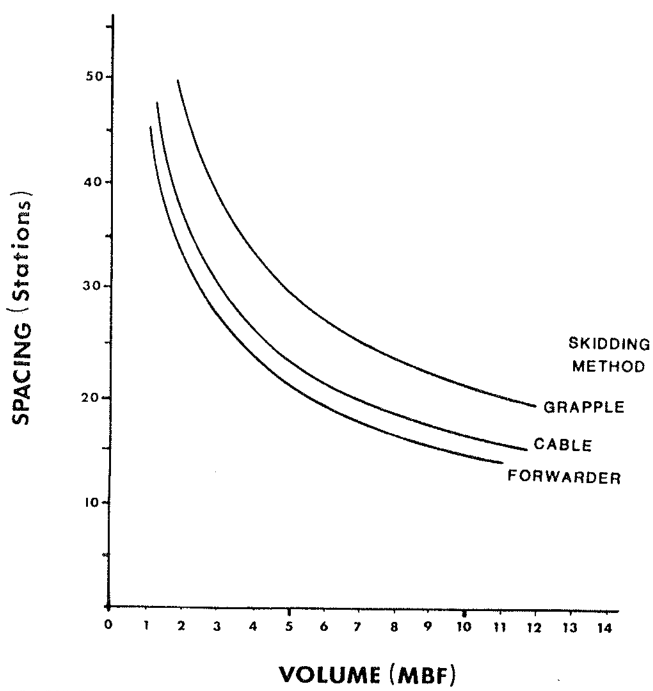

FIGURE 5 Road spacing vs volume for three skidding methods in dry soil (basic model).

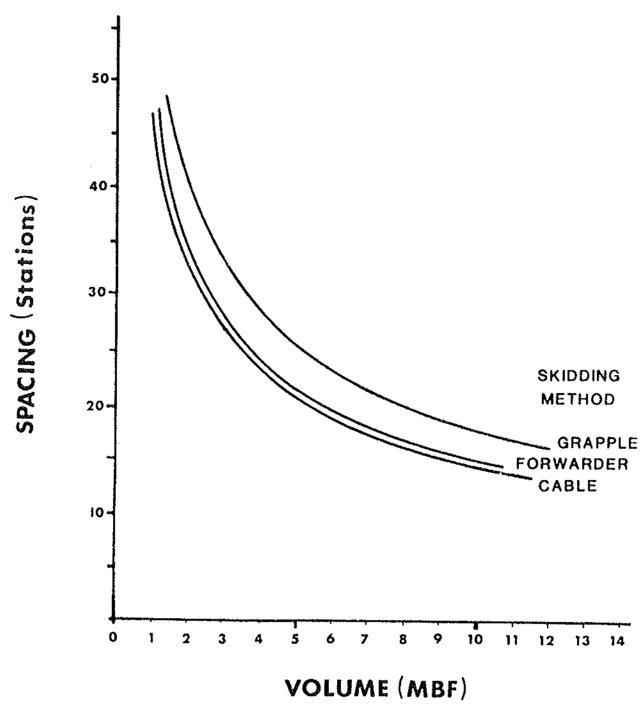

FIGURE 6 Road spacing vs volume for three skidding methods in wet soil (basic model).

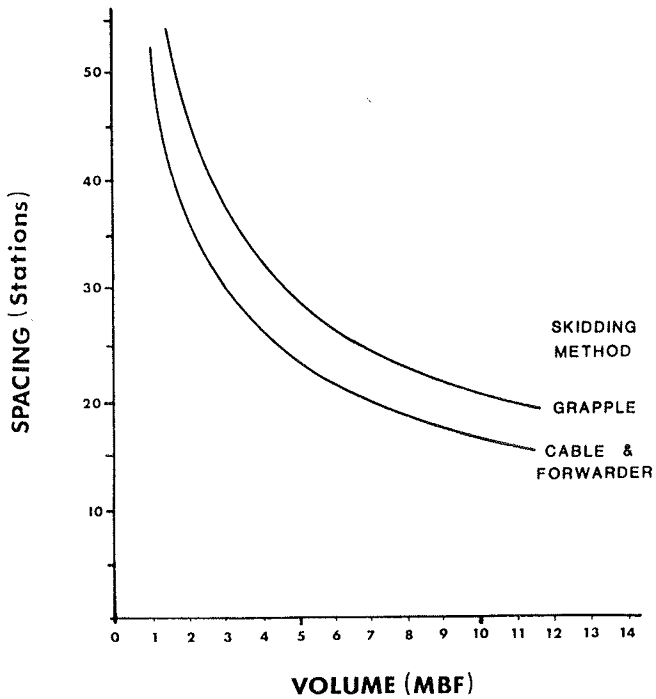

FIGURE 7 Road spacing vs volume for three skidding methods in rocky soil (basic model).

One conclusion that can be drawn from the graphs is that optimum road spacing decreases as timber harvest volume increases for situations in which one road standard on gentle terrain is used. This means that for greater volumes of timber removed, it is more economical to build roads closer together than to skid logs greater distances.

DEVELOPING THE EXPANDED ROAD SPACING MODEL

The basic model was developed for one standard of road situations. The model is based on minimizing the sum of the road construction, skidding, and overhead costs. Two additional road standards and haul costs are included in the expanded road spacing model to make the model more realistic.

A description of a typical forest access system in the Great Lakes States is important to understand the expanded model. Costs considered in this model begin after trees are cut, limbed, and left in tree lengths or sawed into log lengths. Harvesting equipment, such as rubber-tired skidders, forwarders, or crawler tractors, is used to transport logs to the roadside where they are loaded onto logging trucks.

Three standards of roads are used in this expanded model: Class V, Class III, and Class I. These definitions were taken from Table 1 of the *Timber Appraisal Handbook*, which was developed by the U.S. Department of Agriculture, Forest Service (*4*).

Class V roads are built to allow passage of logging trucks. The presence of rocks, roots, and large ruts prevents passage of passenger cars. Forest users, whether commercial, recreational, or administrative, must use light trucks or four-wheel drive vehicles to gain access. Speeds are usually less than 5 mph. Class V roads usually access the timber harvest area from connections to higher standard Class III roads.

Class III roads are more costly than Class V roads. Class III roads follow the natural topography, when possible. They are usually 12-ft-wide, single-lane roads with turn-outs and turn-arounds. Curves are not geometric, but have radii greater than 100 ft. Maximum grades are 15 percent and truck speeds are about 15 mph. Selected aggregates are used to reinforce weak areas of the subgrade. Class III roads are usually passable by passenger cars most of the year. They commonly access timber harvest areas from connections to higher standard Class I roads.

Class I roads, or collector roads, are 18 to 24 ft wide and have minimum curve radii of 400 ft. Grades are less than 8 percent and truck speeds are about 35 mph. Class I roads usually have a crushed gravel surface, and passenger cars can use these roads most of the year. Class I roads are used for recreational and administrative access as well as for timber harvesting access.

The transportation network that provides typical forest access includes all systems used to move the log from the stump to the mill. The log moves from stump to roadside by rubber-tired skidders, forwarders, or crawler tractors. The log then travels from roadside to mill by logging trucks over Class V, Class III, and Class I roads.

An understanding of the transportation route for logs from the stump to the mill aids in an understanding of the development of the expanded road spacing model.

The following assumptions are needed to develop this model:

- The harvest area is on flat terrain.
- The logs are skidded to Class V roads from both sides.
- The timber is distributed evenly over the harvest area.
- Three road standards are used for access (Classes V, III, and I).
- Roads are laid out in a parallel grid pattern and are evenly spaced.

The physical layout of this problem is shown in Figure 8. Following the same logic used in the development of the basic model, Class I and Class III road costs per unit volume are as follows:

Class I road costs per unit volume = $4.356R_1/S_1V$
Class III road costs per unit volume = $4.356R_3/S_3V$

The subscripts identify the class of road being considered.

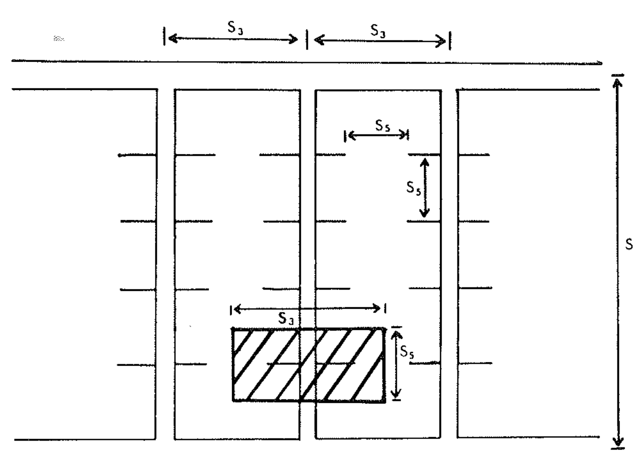

FIGURE 8 Road layout and unit area for the expanded model.

Because the Class V road lengths in the model are not infinite within the unit area, or, in other words, they are dead-ended, the Class V road cost per unit volume relationship will look somewhat different than the relationships developed for Class III and Class I roads.

As in the basic model, a unit area, as shown in Figure 8, results in the following relationship:

Class V road costs per unit volume = $4.356R_5(S_3 - S_5)/S_3S_5V$

Because of the perpendicular relationships between road classes, the cost of hauling along Class V and III roads becomes important in the model. Haul costs are based on distance, travel speed, load size, and truck operation and maintenance. The average haul distance on a Class III road is $S_1/4$, which must be doubled for two-way travel. Therefore, the cost of hauling along Class III roads is as follows:

Haul cost on a Class III road per unit volume
= [2 × (average haul distance (station)) × cost ($-scheduled hour)]/[52.8 (station/mi) × travel speed (mi/scheduled hour) × load size]
= $HS_1/105.6WP_3$

where

H = hourly cost of owning and operating trucks ($/scheduled hour);
P = average travel speed (mi/scheduled hour) (subscript denotes road class); and
W = load capacity of truck in same volume units as V.

Following a similar logic and remembering that Class V roads are not continuous through the unit area defined in the model, it is found that:

Haul cost on a Class V road per unit volume
= $(2 \times ((S_3 - S_5)/4) \times H)/(52.8 \times W \times P_5)$
= $H(S_3 - S_5)/105.6WP_5$

The costs of skidding and overhead are the same as in the basic model: $TS_5F(C + Q)/120U$. At this time all of the road, skid, overhead, and haul cost factors used in the expanded model have been developed and the final total cost equation is as follows:

Total cost per unit volume
$$= 4.356R_1/S_1V + 4.356R_3/S_3V + 4.356R_5(S_3 - S_5)/S_3S_5V + HS_1/105.6WP_3 + H(S_3 - S_5)/(105.6WP_5) + TS_5FC/120U + TS_5FQ/120U$$

The partial derivative of the total cost equation in regard to each road standard defines the spacings that result in minimum cost. This allows the optimum spacing to be determined for each road standard.

$$S_{1opt} = [(4.356R_1/V)/(H/105.6WP_3)]^{1/2}$$
$$S_{3opt} = [(4.356(R_3 - R_5)/V)/(H/105.6WP_5)]^{1/2}$$
$$S_{5opt} = [(4.356R_5/V)/(TF(C + Q)/120U - H/105.6WP_5)]^{1/2}$$

An analysis of the equation for the optimum spacing of Class I roads (S_{1opt}) shows that S_{1opt} is heavily dependent on the cost of hauling along Class III roads and the cost of constructing Class I roads. The optimum spacing occurs at the point where these costs are in balance (Figure 9). The relationship does not contain the factors for skidding and overhead costs, which indicates that the spacing of Class I roads does not depend on these variables.

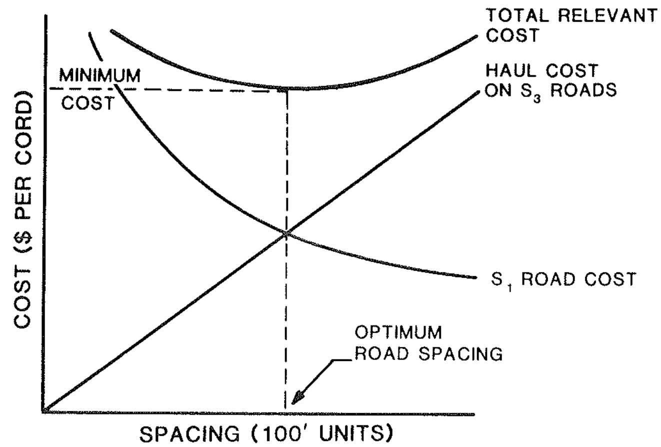

FIGURE 9 Optimizing the spacing of higher standard roads.

Four truck classes and three soil conditions were chosen (Figure 10) to illustrate the effect haul cost has on the relationship for S_{1opt}. The resulting curves (Figures 11, 12, and 13) show how S_{1opt} varies with typical values of these variables. It was assumed that the cost of constructing Class I roads is independent of soil condition.

The equation for the optimum spacing of Class III roads does not contain skidding and overhead costs. This indicates that the spacing of Class III roads does not depend on these variables. Soil condition, however, does affect the spacing of Class III roads, because road construction costs (R_3) are variable costs that depend on soil condition. Typical values are listed for each variable as a function of truck type and soil condition in Figure 14. The type of haul truck used has some effect on the spacing of Class III roads, as shown in Figures 15, 16, and 17. It also appears that slight changes in haul truck speed have a significant effect (4 to 5 stations) on the optimum road spacing.

Finally, an analysis of the equation for optimum spacing of Class V roads (S_{5opt}) shows that optimum spacing depends on the type of haul truck as well as the skidding method. Because skidder operating costs depend on soil condition, the optimum spacing for Class V roads also depends on soil condition.

The effect on Class V road spacing when different kinds of logging trucks are used is shown in Figure 18. This indicates that the haul factor in the optimum road spacing equation for Class V roads (S_{5opt}) is negligible because there is little difference in the value of the optimum spacing between the trucks. This effect can be attributed to the road ends. If Class V roads were extended until they met, the equation for optimum spacing of Class V roads would be exactly the same as for the basic model.

THE COMPUTER PROGRAM

The basic and the expanded optimum spacing models were adapted for use on a microcomputer. The application is menu-

$$\text{Opt. } S_1 = \sqrt{\frac{4.356R_1/V}{H/105.6WP_3}} \qquad R_1 = \$1900/\text{sta}$$

TRUCK \ SOIL	DRY	WET	ROCKY
A Flat bed 4 x 2 Single axle	H = \$30/SH W = 3 MBF P_3 = 14 mph $S_1 = \sqrt{\frac{1{,}220{,}000}{V}}$	H = \$30/SH W = 3 MBF P_3 = 17 mph $S_1 = \sqrt{\frac{1{,}490{,}000}{V}}$	H = \$30/SH W = 3 MBF P_3 = 16 mph $S_1 = \sqrt{\frac{1{,}260{,}000}{V}}$
B Flat bed 6 x 4 Tandem axle	H = \$35/SH W = 4 MBF P_3 = 15 mph $S_1 = \sqrt{\frac{1{,}500{,}000}{V}}$	H = \$35/SH W = 4 MBF P_3 = 19 mph $S_1 = \sqrt{\frac{1{,}900{,}000}{V}}$	H = \$35/SH W = 4 MBF P_3 = 17 mph $S_1 = \sqrt{\frac{1{,}700{,}000}{V}}$
C Truck tractor 4 x 2 Single axle with tandem trailer	H = \$50/SH W = 5 MBF P_3 = 13 mph $S_1 = \sqrt{\frac{1{,}140{,}000}{V}}$	H = \$50/SH W = 5 MBF P_3 = 17 mph $S_1 = \sqrt{\frac{1{,}490{,}000}{V}}$	H = \$50/SH W = 5 MBF P_3 = 15 mph $S_1 = \sqrt{\frac{1{,}310{,}000}{V}}$
D Truck tractor 6 x 4 Tandem axle with tandem trailer	H = \$55/SH W = 6 MBF P_3 = 14 mph $S_1 = \sqrt{\frac{1{,}300{,}000}{V}}$	H = \$55/SH W = 6 MBF P_3 = 18 mph $S_1 = \sqrt{\frac{1{,}700{,}000}{V}}$	H = \$55/SH W = 6 MBF P_3 = 16 mph $S_1 = \sqrt{\frac{1{,}530{,}000}{V}}$

FIGURE 10 Optimum Class I road spacing calculations for four truck types in three soil conditions (expanded model).

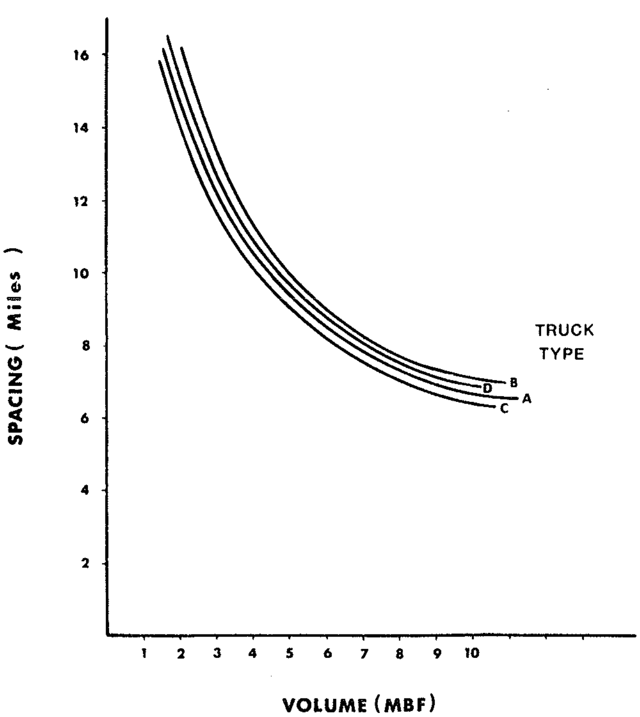

FIGURE 11 Class I road spacing vs volume for four truck types in dry soil (expanded model).

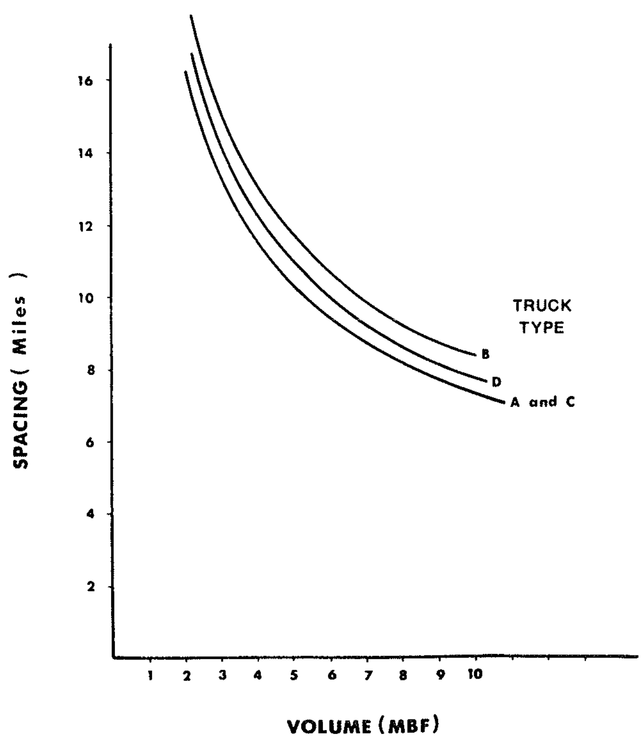

FIGURE 12 Class I road spacing vs volume for four truck types in wet soil (expanded model).

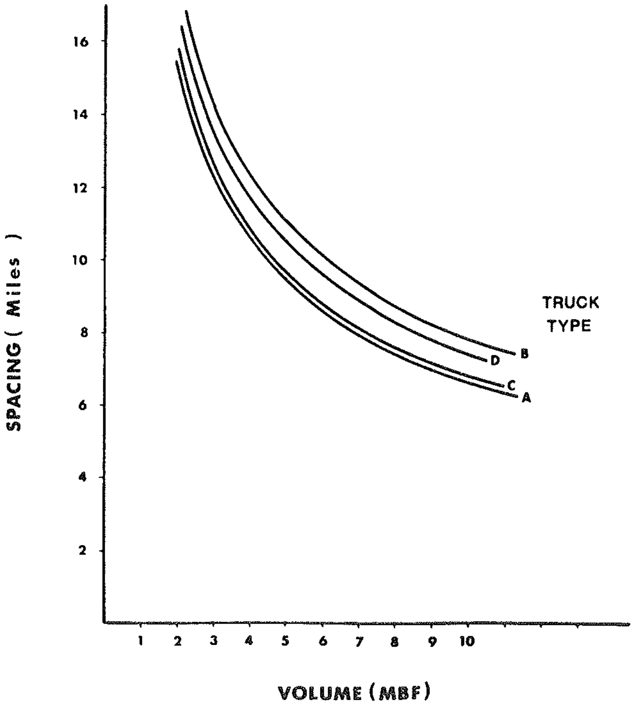

FIGURE 13 Class I road spacing vs volume for four truck types in rocky soil (expanded model).

driven and interactive; descriptive prompts are provided for all data elements. Optimum road spacing can be computed and displayed for single or ranges of volume estimates. Users can vary any of the dependent factors and instantly see the effects on optimum road spacing.

The software for the model is a LOTUS 1-2-3 template (*5*). Any microcomputer capable of running LOTUS 1-2-3 can use this program. Knowledge of spreadsheet operation is not necessary, although spreadsheet experience would aid in further exploration of the model.

DISCUSSION OF THE MODEL

The expanded optimum road spacing model developed here provides a foundation for further theoretical development. Potential application exists for any harvesting situation on gentle terrain that requires road access to gather a uniformly distributed commodity and haul it to a final destination. Further development of the model could make it applicable to situations other than timber harvest.

Many opportunities exist to expand on this model. Overhead costs are incorporated into this work. They are just one of the many possible factors and costs that could be used to expand the model and make it more relevant to individual situations.

Opt. $S_3 = \sqrt{\dfrac{4.356(R_3-R_5)/V}{H/105.6WP_5}}$ R_5 = \$125/sta

TRUCK \ SOIL	DRY	WET	ROCKY
A FLAT BED 4 X 2 SINGLE AXLE	R_3 = \$300/Sta H = \$30/SH W = 3 Mbf P_5 = 4.5 mph $S_3 = \sqrt{\frac{36225}{V}}$	R_3 = \$435/Sta H = \$30/SH W = 3 Mbf P_5 = 2.5 mph $S_3 = \sqrt{\frac{35650}{V}}$	R_3 = \$400/Sta H = \$30/SH W = 3 Mbf P_5 = 3.5 mph $S_3 = \sqrt{\frac{44275}{V}}$
B FLAT BED 6 X 4 TANDEM AXLE	R_3 = \$300/Sta H = \$35/SH W = 4 Mbf P_5 = 5.0 mph $S_3 = \sqrt{\frac{46000}{V}}$	R_3 = \$435/Sta H = \$35/SH W = 4 Mbf P_5 = 3.0 mph $S_3 = \sqrt{\frac{48890}{V}}$	R_3 = \$400/Sta H = \$35/SH W = 4 Mbf P_5 = 4.0 mph $S_3 = \sqrt{\frac{57830}{V}}$
C TRUCK TRACTOR 4 X 2 SINGLE AXLE WITH TANDEM TRAILER	R_3 = \$300/Sta H = \$50/SH W = 5 Mbf P_5 = 4.0 mph $S_3 = \sqrt{\frac{32200}{V}}$	R_3 = \$435/Sta H = \$50/SH W = 5 Mbf P_5 = 2.0 mph $S_3 = \sqrt{\frac{28520}{V}}$	R_3 = \$400/Sta H = \$50/SH W = 5 Mbf P_5 = 3.0 mph $S_3 = \sqrt{\frac{37950}{V}}$
D TRUCK TRACTOR 6 X 4 TANDEM AXLE with TANDEM TRAILER	R_3 = \$300/Sta H = \$55/SH W = 6 Mbf P_5 = 4.5 mph $S_3 = \sqrt{\frac{39520}{V}}$	R_3 = \$435/Sta H = \$55/SH W = 6 Mbf P_5 = 2.5 mph $S_3 = \sqrt{\frac{38890}{V}}$	R_3 = \$400/Sta H = \$55/SH W = 6 Mbf P_5 = 3.5 mph $S_3 = \sqrt{\frac{48300}{V}}$

FIGURE 14 Optimum Class III road spacing calculations for four truck types in three soil conditions (expanded model).

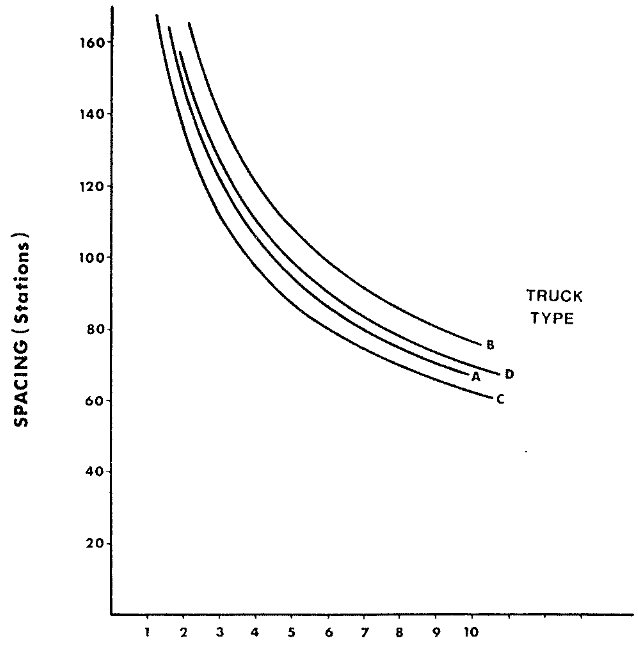

FIGURE 15 **Class III road spacing vs volume for four truck types in dry soil (expanded model).**

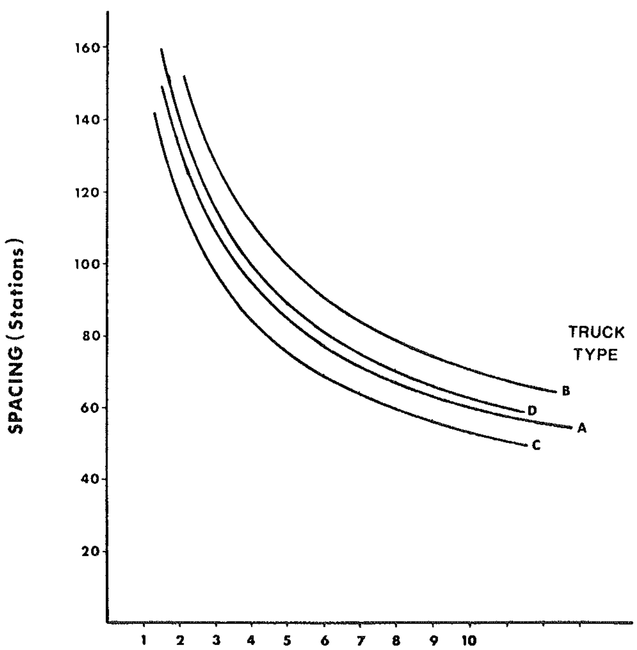

FIGURE 16 **Class III road spacing vs volume for four truck types in wet soil (expanded model).**

As was stated in the assumptions, the model assumes that all skidding will be to Class V roads regardless of the proximity of the Class III roads to the harvest area. Class III roads are spaced rather far apart: therefore, this assumption will not significantly influence the results.

In addition, the assumed average skid distance is actually somewhat higher than $S_5/4$ because of the influence of the road

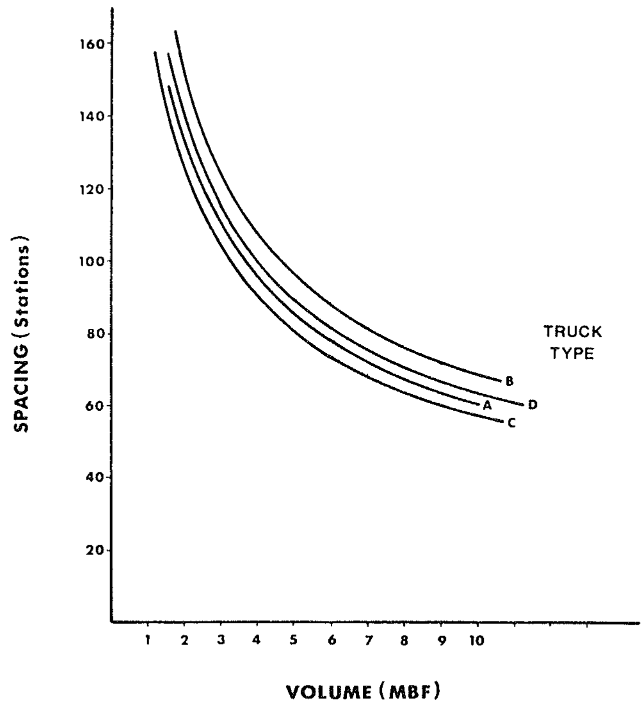

FIGURE 17 **Class III road spacing vs volume for four truck types in rocky soil (expanded model).**

end on average skid distance. This effect is included in the factor for sinuosity.

Some problems exist with the model. Assumptions that were made to facilitate development of the model could make its application unrealistic in some situations. Users are reminded that the model serves as a guide in the development of a harvest plan and provides information that can greatly minimize the total costs of the harvest and hauling operation. The model, however, cannot be substituted for subjective decisions regarding environmental, political, and social effects.

One might infer from some of the figures that the skidding or haul method that shows the closest spacing would have the least-cost method of skidding or hauling. This is not necessarily the case. These curves define a minimum-cost situation that relates only to road spacing. They cannot be used to make cost comparisons relative to any other factors. The user should be aware that optimum spacing does not mean optimum costs relative to each individual factor, but only relative to spacing.

A brief analysis of converting the model's optimum road spacing for an area to road density indicated that caution is required to use the model in this manner. The road density approach does not assure an even road spacing, as assumed in the model. Uneven road spacings cause the average skid distance to vary. The calculated optimum road spacing will not be accurate if the average skid distance is not accurate. Therefore, it is not recommended that the results from the road spacing model be converted to road densities.

SUMMARY

A basic model for road spacing was developed from Matthew's theory of balancing road and skidding costs to minimize the sum of these costs. Examples in which the basic model was used

$$\text{Opt. } S_5 = \sqrt{\frac{4.356R_5/V}{\frac{TF(C+Q)}{120\ U} - \frac{H}{105.6WP_5}}}$$

Haul truck	A Flat bed 4 x 2 Single axle H = \$30/SH W = 3 MBF P_5 = 4.5 mph	B Flat bed 6 x 4 Tandem axle H = \$35/SH W = 4 MBF P_5 = 5 mph	C Truck tractor 4 x 2 Single axle with tandem trailer H = \$50/SH W = 5 MBF P_5 = 4 mph	D Truck tractor 6 x 4 Tandem axle with tandem trailer H = \$55/SH W = 6 MBF P_5 = 4.5 mph
Optimum S_5	14.2 Stations	14.1 Stations	14.3 Stations	14.2 Stations

R_5 = \$125/sta
U = 0.95 MBF
V = 10 MBF/acre

C = \$50/SH
Q = \$50/SH
T = 0.3 min/sta
F = 1.10

FIGURE 18 Comparison of optimum Class V road spacing for four truck types (grapple skidder in dry soil).

were analyzed and it was concluded that road spacing is affected by soil condition, equipment operation and maintenance, and timber volumes. This model can be effective in determining an optimum balance between road spacing and skidding.

The basic model was expanded to make it more relevant and applicable to more situations. Haul costs and two additional road standards were incorporated into the model. Analysis of the expanded model showed that total relevant cost was most sensitive to the spacing of the lowest standard of road.

In summary, the spacing of low-volume, low-standard roads can greatly affect the layout and the total cost of a harvesting operation. The application of the model to a microcomputer provides an easy method to analyze the effects of the variable factors. This information may help to minimize the combined costs of harvesting and road building.

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The New Zealand Vehicle Operating Costs Model

CHRISTOPHER R. BENNETT

The development of a computer model to predict vehicle operating costs for use in highway economic appraisals is described. Like many smaller, less wealthy countries, New Zealand did not have the capital or resources available to conduct its own research into vehicle operating costs. After an extensive literature review, it was decided to adopt the World Bank's HDM-III Brazil operating cost relationships and an Australian model for passenger car and light commercial vehicle fuel consumption. Depreciation costs were calculated using the results of a New Zealand study that successfully differentiated between the use and age-related components. A computer model was developed that used these relationships to predict vehicle operating costs. It was used to prepare tables of costs for use in manual appraisals and to analyze individual segments of roads. It employs a three-zone, speed-volume model to consider traffic interactions on individual segments of road. For all calculations, the model uses standardized distributions of vehicle speeds. These associate a range of speeds with a mean value to calculate a composite cost that reflects the entire range of speeds of all vehicles traveling in the traffic stream.

Low-volume roads pose a difficult problem for highway authorities throughout the world. Whereas improvements to roads that carry high volumes usually accrue substantial benefits, the benefits for low-volume roads are relatively low and the economic justification of improvements is often marginal. It is therefore essential that all investments in low-volume roads be screened so that only the economically feasible projects are allowed to proceed. Ideally, screening should be achieved by a rigorous economic appraisal that compares the project costs with the benefits in the form of travel time savings and reduced vehicle operating costs.

HIGHWAY ECONOMIC APPRAISALS IN NEW ZEALAND

New Zealand is a small country with a population of 3.3 million and a land area of 268,105 km^2. With a Gross Domestic Product of approximately $7,000 (U.S.) per capita, New Zealand is one of the least wealthy of the Organization of Economic Co-operation and Development countries. The major export industries are related to primary produce. The rural sector is serviced by some 80,000 km of roads and there are approximately 18,000 km of urban roads.

Most of the population is centered in the major coastal cities; the majority of the rural roads therefore carry low volumes. In a recent study of the roading network, it was estimated that the average traffic on 26,000 km of sealed roads was 320 vehicles per day (vpd) and 76 vpd on 42,000 km of unsealed roads (*1*).

With such a dispersed network supported by a small population and limited resources, the roading authorities recognized the importance of economic appraisals quite early. Economic appraisals were first discussed in the 1940s in the context of the effects of different alignments on operating costs and maintenance policies (*2*). In the late 1960s they became widely accepted and in 1971 the first standard techniques for highway economic appraisals were introduced (*3*).

The use of economic appraisals has increased rapidly and it is now required that all projects receiving funds from the Treasury be shown to be economically viable. This has led to the recent publication of a Technical Recommendation for the Economic Appraisal of Roading Improvement Projects called TR9 (*4*). This publication provides a relatively simple manual method for conducting economic appraisals. It also provides curves and tables of vehicle operating costs that can be used in the appraisals.

The Prediction of Vehicle Operating Costs for Economic Appraisals

The most important element of an economic appraisal is the accurate prediction of the vehicle operating costs. In preparing TR9 it was decided that it would be necessary to employ operating cost relationships that were developed overseas because insufficient time or funds were available to develop such relationships within New Zealand. This situation is common in many of the less wealthy countries that have insufficient capital or resources available to conduct their own research. Fortunately, the literature abounds with the results of research into predicting vehicle operating costs. The main difficulty therefore lies in selecting the most appropriate relationships for the country in question.

It is possible to broadly categorize the research into that which was conducted before the 1970s and that which was conducted since then. Research into motor vehicle fuel consumption began in the 1920s and the major emphasis was on the advantages of gravel and paved surfaces over dirt surfaces (*5*). It was not until the 1950s, however, that research into all aspects of vehicle operating costs became common, and in the late 1960s and early 1970s a number of works were published that compiled the operating costs that were related to road design standards. A summary of the early works and a description of the data bases on which they were based can be found elsewhere (*6*).

A number of deficiencies are associated with this early work. Several authors used personal judgment to supplement data bases that were of questionable accuracy. Claffey used direct experiments augmented by surveys, which makes his study the

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most reliable of the early works; however, the vehicles that were used bear little resemblance to the modern vehicle fleet (*7*). In spite of these deficiencies, a number of authorities still use these early, outdated operating cost estimates for economic appraisals.

In 1969 the World Bank initiated a program to investigate the interrelationship between construction, maintenance, and vehicle operating costs on low-volume roads. Because of the lack of reliable information, a major user cost study was undertaken in Kenya from 1971 to 1975 that investigated paved and unpaved road deterioration and factors that affect vehicle operating costs (*8, 9*). User cost studies were later conducted in the Caribbean, India, and Brazil (*10-13*).

The results of these studies have been incorporated by the World Bank into their HDM-III model (*14*). The World Bank staff has also reanalyzed much of the Brazil study to develop modified Brazil relationships specifically for use in the model. The HDM-III model is a comprehensive macroscopic model that represents the state of the art in economic appraisals of low-volume roads. It can be used to optimize highway investments on a national level. However, because of its complexity, it is unsuitable for use in microscopic evaluations of the type that are undertaken in New Zealand.

It was therefore decided to undertake the development of a model for use in New Zealand that was based on HDM-III but tailored more to New Zealand's small-scale requirements. The model would also be used to prepare tables and curves of vehicle operating costs that could be used in manual appraisals. This led to the development of the New Zealand Vehicle Operating Costs (NZVOC) model.

THE NEW ZEALAND VEHICLE OPERATING COSTS MODEL

The NZVOC model was developed at the University of Auckland (*15*). After the operating cost relationships in HDM-III and others available in the literature were evaluated, it was recommended that the HDM-III Brazil relationships be used in New Zealand to predict all operating costs except passenger car and light commercial vehicle fuel consumption. It was believed that Caribbean relationships would be more appropriate for these categories. In addition, depreciation relationships were developed from a study of New Zealand vehicles and a modified Transportation Road Research Laboratory (TRRL) methodology was used to estimate the costs (*15*).

The Brazil and Caribbean relationships were selected after considering the data bases from which they were developed, the operating conditions in the original studies, and the relevance of the study vehicles to New Zealand. This type of an evaluation should be undertaken whenever overseas relationships are being considered for use because only then can it be ensured that the resulting operating costs are relevant and can be used with a measure of confidence.

For the TR9 project, the NZVOC model was rewritten and enhanced to form the NZVOC2 model. In the NZVOC2 model, the Caribbean relationships were replaced by relationships that were developed in Australia because these were found to be more applicable for use in New Zealand. The same relationships that were in the original NZVOC model were used for the other cost components.

The features of the NZVOC2 model and examples of its predictions are described in the following sections. A complete discussion of the model's structure, operating cost relationships, and input data can be found elsewhere (*16, 17*).

Representative Vehicles

The NZVOC2 model considers six classes of representative vehicles: passenger cars, light commercial vehicles, medium commercial vehicles, two groups of heavy commercial vehicles (HCV-I and HCV-II), and buses. This system was selected by considering the nature of the New Zealand vehicle fleet and the available operating cost data.

The use of six vehicle classes provides analysts with more flexibility in calculating the operating costs for different traffic conditions. It is important that operating costs reflect the costs of the entire vehicle class; this is achieved by basing them on the costs for a number of different vehicles within the class. The costs for the individual vehicles are then aggregated into a composite cost that can be considered to be representative of all vehicles in the class. A total of 15 representative vehicles are used in the modeling; they are summarized in Table 1 (*17*).

Unit Costs

One of the most useful features of the more recent operating cost relationships is the way in which they estimate vehicle operating costs in terms of the consumption of resources. This consumption is multiplied by the costs of the different components to obtain the financial or economic cost. When preparing standard estimates of vehicle operating costs, it is therefore possible to quickly revise the costs to reflect changes in the costs of the individual components. This is a useful feature in countries with high inflation rates in which cost estimates should be frequently updated. It also means that it is not necessary to resort to the use of inflation indices for the cost updating. These indices often reflect changes in costs outside of the transport sector. The values of the economic costs of the various components that were used in the TR9 project are summarized in Table 2 (*18*).

Vehicle Operating Costs

As was discussed earlier, the NZVOC2 model draws mainly on the World Bank HDM-III model for its vehicle operating cost relationships. The relationships used to predict vehicle speeds and the various operating cost components are summarized in Table 3.

It is impossible here to reproduce the actual relationships in their entirety. Readers interested in the form of the relationships should refer elsewhere for a complete discussion of their theoretical development (*19*). However, a discussion follows of the nature of the relationships and why they were selected for use in New Zealand. The Brazil models discussed are those developed by the World Bank for the HDM-III model and are not the original relationships developed in Brazil (13, 14).

Speed

Because economic appraisals generally consist of evaluating the effects that the alteration of roadway characteristics will have

TABLE 1 REPRESENTATIVE VEHICLES USED IN THE NZVOC2 MODEL

Vehicle Class	Vehicle	Type of Fuel	Percentage of Vehicles
Passenger car	Small passenger car	Petroleum	82.0
	Medium passenger car	Petroleum	18.0
Light commercial	Light van/utility	Petroleum	50.0
	Medium van/utility	Petroleum	35.0
	Light truck/heavy utility	Petroleum	10.0
	Light truck/heavy utility	Diesel	5.0
Medium commercial	4-tonne truck	Petroleum	7.0
	4-tonne truck	Diesel	26.0
	6-tonne truck	Diesel	34.0
	8-tonne truck	Diesel	33.0
Heavy commercial (HCV-I)	15-tonne, 3-axle truck	Diesel	100.0
Heavy commercial (HCV-II)	Multi-axle/artic. truck	Diesel	75.0
	Multi-axle/artic. truck towing	Diesel	25.0
Bus	Urban bus	Diesel	50.0
	Rural bus	Diesel	50.0

TABLE 2 UNIT COSTS USED IN 1986 COST CALCULATIONS[a]

Vehicle Class	New Vehicle ($/veh)	New Tire ($/tire)	Labor ($/hr)	Petroleum ($/liter)	Diesel ($/liter)	Engine Oil ($/liter)
Passenger car	14,100	125	27	0.6697	-	3.558
Light commercial	14,100	125	27	0.6697	-	3.558
Medium commercial	32,800	225	27	0.6697	0.5848	3.558
Heavy commercial (HCV-I)	101,051	410	27	-	0.5848	3.558
Heavy commercial (HCV-II)	143,400	410	27	-	0.5848	3.558
Urban bus	140,600	410	27	-	0.5840	3.558
Rural bus	231,800	410	27	-	0.5848	3.558
Trailer	30,400	410	27	-	-	-

Notes: Dash = no cost applicable. All costs in New Zealand dollars ($1.00 New Zealand = $0.55 U.S.) as of April 1, 1986

TABLE 3 NZVOC2 MODEL VEHICLE OPERATING COST RELATIONSHIPS

Vehicle Class	Component Speed	Fuel	Tires	Oil	Parts	Labor	Depreciation
Passenger car	Brazil[a]	ARRB[b]	Brazil	HDM-III[c]	Brazil	New Zealand	New Zealand
Light commercial	Brazil	ARRB	Brazil	HDM-III	Brazil	New Zealand	New Zealand
Medium commercial	Brazil	Brazil	Brazil	HDM-III	Brazil	New Zealand	New Zealand
Heavy commercial (HCV-I)	Brazil	Brazil	Brazil	HDM-III	Brazil	New Zealand	New Zealand
Heavy commercial (HCV-II)	Brazil	Brazil	Brazil	HDM-III	Brazil	New Zealand	New Zealand
Bus	Brazil	Brazil	Brazil	HDM-III	Brazil	New Zealand	New Zealand

[a]Relationships developed by the World Bank from Brazil study data not GEIPOT relationships.
[b]Australian Road Research Board relationships.
[c]Relationships developed for use in World Bank HDM-III model.
[d]New Zealand developed or modified relationships.

on time and operating costs, it is necessary to use a method by which the impact of these changes on vehicle speeds can be quantified. This is not only useful in an economic appraisal, but it can be used to evaluate the optimum, or preferred, alignment during the planning stages.

The Kenya, Caribbean, and India studies developed linear regression models that related the effect of geometry on vehicle speeds. A typical relationship from Kenya is as follows (*8*).

$$V = 102.6 - 0.372RS - 0.076F - 0.111C - 0.0049A \quad (1)$$

where

V = mean speed of cars in km/hr,
RS = average rise in m/km over the road segment,
F = average fall in m/km over the road segment,
C = aggregate curvature in degrees/km, and
A = altitude in m above sea level.

A complete set of equations is given in a study by Bennett in which their predictions are discussed and compared (*15*).

Regression equations of this nature have two major shortcomings (*20*). An inconsistency exists in the coefficients between the equations from the various studies, which is partly a result of the high degree of correlation between road characteristics. Curves and grades are often found together and the regression procedure may not correctly isolate the independent effects of curves and grades.

The second shortcoming is that the regression equations are unable to handle extreme conditions. It is possible to obtain negative values of speed for reasonable combinations of road characteristics. Furthermore, the geometric effects have a constant effect regardless of the values of other geometric features. This means, for example, that speeds on a steep upgrade could be increased by making the surface smoother even though the gradient is the speed-limiting factor and not the surface condition (*20*).

The World Bank developed an alternative method of modeling speed-geometry effects for the HDM-III model using data from the Brazil study (*19*). The method assumes that each vehicle has a set of limiting maximum speeds for open roads, upgrades, downgrades, curves, and rough surfaces. At any given point, the speed will be the minimum of these five values.

The Brazil model provides equations that relate each of these limiting speeds to road geometry and roughness characteristics. The model is based on mechanistic principles and requires data that detail vehicle mass, load, frontal area, and aerodynamic drag. It also uses a number of vehicle and driver parameters that were quantified from the field data of the Brazil study.

The effects of grade in the Brazil speed prediction model are depicted in Figure 1. At low grades, the steady state speed is governed by the psychological, roughness, and curve constraints. As the grades become more pronounced, the effects of the other constraints lessen until the only constraint is that which results from the gradient.

The Brazil model is conceptually more attractive than the simple linear models because it is based more firmly on principles of driver and vehicle behavior. Concerns have been expressed about the techniques used to quantify the model parameters; however, it still constitutes the best speed prediction model currently available (*20*).

Before the model could be applied in New Zealand, it was necessary to calibrate its predictions. This should ideally take the form of requantifying all regression-based model parameters. Because such an undertaking was beyond the scope of the TR9 project, it was assumed that the effect of geometry on vehicle speeds observed in Brazil would apply in New Zealand and that therefore the regression-based values would be applicable. Therefore, only the desired speed constraint was changed so that the Brazil model would give predictions similar to what were observed on flat, straight roads with average roughness levels in New Zealand.

Fuel Consumption

Both the Australian Road Research Board (ARRB) and the Brazil fuel consumption models are based on mechanistic principles and have the same underlying structure (*21*). The ARRB model was selected for passenger cars and light commercial vehicles in preference to other models because the vehicles on which it is based are more similar to what are found in New Zealand. Its predictions can also take urban traffic interactions into consideration. Accordingly, TR9 contains two

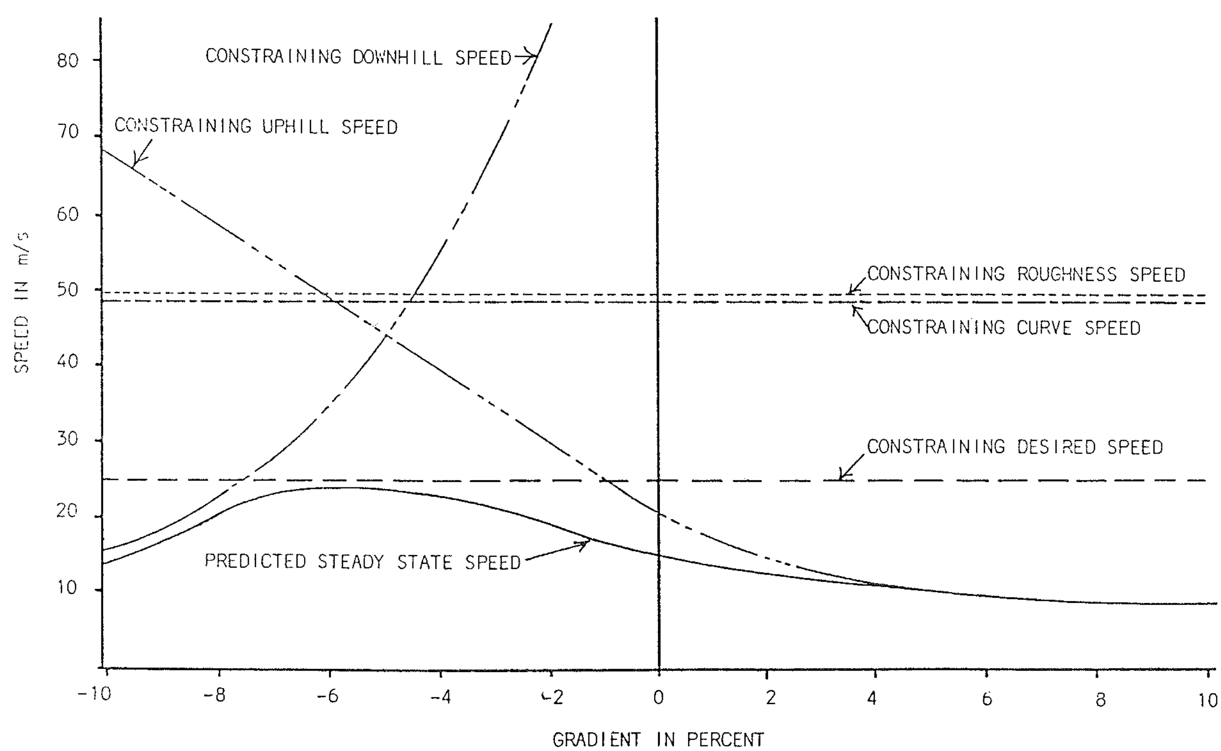

FIGURE 1 Constraining and steady-state speeds versus gradient.

sets of fuel costs for passenger cars and light commercial vehicles; one is for urban conditions and the other is for rural, free-flowing conditions.

The Brazil relationships are the only mechanistic models available for predicting the fuel consumption of a range of commercial vehicles. Unfortunately, technological changes to vehicles since the mid-1970s may have rendered these models obsolete for some applications. Because the average service life of commercial vehicles is approximately 15 years, this is not considered to be a significant problem in New Zealand (*15*). However, for countries with low vehicle service lives, use of these models may result in an overprediction of fuel consumption.

The accuracy of the fuel consumption models was evaluated on a macroscopic level in another study (*1*). When using annual traffic volume data for all of New Zealand, it was found that the predicted petrol consumption calibrated to within 5 percent of the observed national petrol consumption. The diesel consumption calibrated to within 63 percent. However, a large segment of the difference was ascribed to an inability to accurately specify the commercial vehicle traffic composition. Overall, the models were considered to give a good representation of motor vehicle fuel consumption.

Tire Consumption

The Kenya and Caribbean studies both developed tire consumption relationships that employed roughness as an independent variable. Vehicle mass was employed as an independent variable for trucks. The India study relationships related tire consumption to a wide range of road characteristics, including width, curvature, roughness, and gradient. For cars and light commercial vehicles, the Brazil tire consumption model is a simple, linear relationship with roughness as the independent variable. A more complex model that is based on mechanistic principles has been developed for trucks and buses.

It was decided to adopt the Brazil model to predict passenger car tire consumption. The India model was considered to be unsuitable because it was believed that the New Zealand road and traffic characteristics were significantly different from those encountered in the India study. The Caribbean study did not include observations at low roughnesses, which are commonly encountered in New Zealand. The Kenya model was rejected because the tire lives appeared unreasonably high in the New Zealand context. The Brazil model was selected for commercial vehicles because its mechanistic basis made it superior to those of the other studies.

There are two components to tire consumption: tread wear and carcass wear. The Brazil mechanistic model predicts tread wear as a function of the wearable volume of rubber and the forces that act on the tire. Carcass wear is considered in terms of the number of retreads that the carcass may be suitable for, which is affected by the road roughness and curvature.

In developing the model, it was found that insufficient data existed to quantify the effects of lateral forces on tire wear (*19*). This is a serious shortcoming of the model. Lateral forces are only considered through a very approximate relationship between superelevation and curvature. A second deficiency in the model may be in the assumptions employed in considering retreaded tires. The findings of the Brazil study are in conflict with those of other studies (*15, 20*).

In spite of these shortcomings, the Brazil mechanistic model offers a significant advancement over the other regression-based models. Because it considers the effects of forces on tire wear, it is the most versatile model and, therefore, the most readily transferable between countries.

Oil Consumption

The costs of oil and lubricants are calculated using the HDM-III model (*14*). This model was developed after considering the results of a number of studies on the effects of operating conditions on oil consumption. It consists of a base rate of oil consumption and a linear term that is a function of surface roughness. The constant of the linear term is surprisingly the same for all vehicle types even though the base rates vary significantly.

It was believed that the model gave predictions for passenger cars and light commercial vehicles that were excessive in the New Zealand context. This difference is probably a result of the type of light vehicles found in New Zealand and local maintenance practices. The base rates of consumption for these vehicles were reduced by 1.0L/1000 km to rectify the situation.

Parts Consumption and Labor Costs

Particular care must be taken when applying parts and labor models that were developed in different countries. Maintenance and repair costs are affected by management decisions and operating conditions. Therefore, it can be anticipated that significant variation will exist in these costs not only between countries but perhaps between different regions of the same country. Labor costs will show the greatest variation with location; so calibration exercises should therefore focus on these costs.

The Kenya, Caribbean, and Brazil parts consumption models use road roughness and distance traveled as independent variables. The India study related parts consumption to these and other variables that reflect roadway geometry.

The India relationships were excluded from consideration for use in New Zealand because the traffic stream in India is heterogeneous and is composed of a range of vehicles, from bullock carts to auto-rickshaws. These conditions make the India relationships unsuitable to countries with significantly different traffic conditions.

The Caribbean model is not based on observations of the relatively low levels of roughness that are commonly found in New Zealand. As illustrated in another study, this factor, and the nature of the model structure, result in unreasonable predictions. The Kenya passenger car and light commercial vehicle relationships are also not based on observations at low roughness levels, although the range of roughness considered encompasses the majority of roads in New Zealand.

The only parts consumption model that encompasses the complete range of roughness levels found in New Zealand is the Brazil model, and it is also based on an extremely comprehensive data base. It predicts that the impact of roughness on parts consumption will generally increase with increasing roughness, which is a logical response. It was therefore decided to adopt the Brazil parts consumption model for use in New Zealand.

Because of the concerns discussed earlier in regard to the transferability of the labor models, it was decided not to employ a labor model that was developed in one of the user cost studies. It has been suggested that the ratio of parts to labor costs lies in the range of 50:50 to 60:40 (*22*). The approach adopted was to have a constant cost based on a 55:45 split between parts and labor costs. This value was selected because parts in New Zealand comprise a slightly larger portion of the total maintenance and repair costs than labor.

Unlike the other component costs, estimates of maintenance and repair costs based on New Zealand surveys were available. It was clearly desirable to calibrate the Brazil parts consumption model so that its predictions were the same as these known values at average roughness values. Two methods were available for calibrating the model; the model parameters could be modified or a constant term, which may be negative, could be added to the predictions. As was illustrated in another study, the latter is the only appropriate method, because it does not affect the impact of the independent variables on the model predictions (*15*).

The parts model was therefore calibrated by first determining the parts component of the New Zealand costs based on the 55:45 split between parts and labor costs. A constant term was then added to the Brazil model predictions so that they gave the same values at an average roughness level.

Depreciation

The depreciation expense of a motor vehicle arises as a result of usage, age, and technological obsolescence. The use-related depreciation should be treated as a running cost. The age and technological obsolescence costs are overhead costs because they occur independently of vehicle use. For TR9, the depreciation costs are calculated using the relationships and methodology established in another study (*15*). It was found that passenger car and commercial vehicle depreciation could be predicted using relationships of the following form:

Passenger cars: $DEP = C\,(AGE)^{A}\,(KILOM)^{B}$ (2)

Commercial vehicles: $DEP = C - D\,(AGE)^{A}\,(KILOM)^{B}$ (3)

where

DEP	=	depreciation of a vehicle as a percentage of the vehicle replacement price,
AGE	=	age of vehicle in years,
$KILOM$	=	cumulative kilometrage of vehicles, and
A to D	=	constants.

These depreciation relationships were established using the following methodology:

- Data on recent resale values of motor vehicles, and their age and distance traveled, were collected from newspapers and dealers' guides.
- The original retail value of vehicles was determined and inflated into current dollars.
- The depreciation (as a percentage) is the difference between the inflated original value and the recent resale value.
- An SAS nonlinear regression package was used to relate the vehicle depreciation to its age and distance traveled (constants A to D in Equations 2 and 3) (*23*).

The use of these relationships in conjunction with the age and cumulative kilometrage data enable the value of a vehicle at any stage of its life to be calculated. These data can then be aggregated to provide a single value to express depreciation costs for the fleet. This process can be mathematically expressed as follows (*17*):

$$DEPCOF = \sum_{i=1}^{n} FREQ_i \; CDEP_i \qquad (4)$$

where

$DEPCOF$	=	depreciation coefficient,
$FREQ_i$	=	percentage of vehicle of age i in the fleet, and
$CDEP_i$	=	change in depreciation over year i.

The variable $CDEP_i$ is defined as follows:

$$CDEP_i = DEP_i - DEP_{i-1} \qquad (5)$$

The annual depreciation costs are obtained by multiplying the depreciation coefficient by the replacement price of a vehicle.

The weighting of the exponents in the depreciation relationships gives the relative percentages of the total depreciation that result from use and age. The following are percentages of the total depreciation that results from vehicle use: passenger cars—40 percent, light commercial vehicles—30 percent, other commercial vehicles—20 percent, and buses—20 percent.

Vehicle Speeds

Economic appraisal techniques generally have not adequately considered the sensitivity of vehicle operating costs, particularly fuel costs, to operating speeds. The standard practice is to estimate the mean speed of the traffic stream using models such as those discussed earlier and then calculate the vehicle operating costs that correspond to this speed.

This practice ignores the fact that the average speed of vehicles on a road is composed of a distribution of speeds. The costs that correspond to each speed in the distribution should be calculated and then combined to form a composite speed cost for a stream of vehicles with a given mean speed. This composite cost can be calculated using the following relationship (*17*):

$$COMCOST = \sum_{i=1}^{n} COST_i \; SPFR_i \qquad (6)$$

where

$COMCOST$	=	the composite speed costs for a stream of traffic with a mean speed of V,
$COST_i$	=	the cost of a vehicle traveling at speed i,
$SPFR_i$	=	the number of vehicles in the stream traveling at speed i, and
n	=	the number of different speeds in the distribution.

As was discussed in another study, when individual spot speed observations in a sample are divided by the sample mean

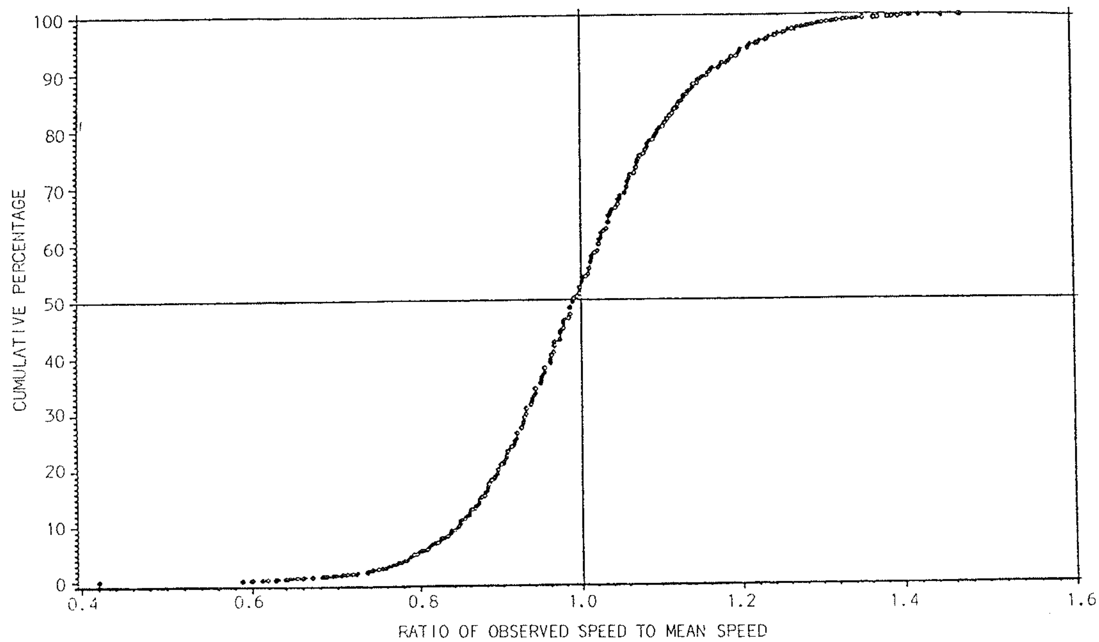

FIGURE 2 Standardized passenger car speed distribution.

speed, the resulting standardized distribution is identical to standardized distributions from other sites (*24*). Data from six spot speed studies that were conducted throughout New Zealand were used to establish standardized distributions for nine different vehicle classes. Figure 2 is an example of the standardized passenger car distribution (*24*).

Standardized distributions make it possible to fully describe the speed distribution solely from the mean speed. They provide details of the the range of speeds associated with the mean value and also of the number of vehicles that can be found over the range. Given a mean speed, which can either be predicted using the Brazil speed prediction model or estimated by the user, the NZVOC2 model uses the appropriate standardized speed distribution for the vehicle class to calculate the composite speed costs. This results in differences in fuel costs of up to 10 percent over those associated with the mean speed.

Speed-Volume Effects

Speed-volume effects are an important consideration in economic appraisals. Traffic delays on rural roads are usually associated with the supply and demand of overtaking opportunities. The demand often cannot be met and vehicles are forced to travel at the speed of the slowest vehicle until a passing opportunity presents itself.

The literature is summarized in another study that suggests that speed-volume effects for one vehicle type depend on the free speeds of other vehicle types (*20*). It goes on to suggest the use of a three-zone speed-volume relationship such as is shown in Figure 3. Below a certain volume, *Q0*, there are no interaction effects. Between this volume and the practical capacity, *QCAP*, speeds decrease as a result of the interaction effects. At the practical capacity, all vehicles travel at the speed of the slowest type. Beyond the practical capacity, the speeds decrease rapidly to a minimum jam speed, which is reached at maximum congestion. This process can be mathematically expressed as follows (*20*):

$$SQ_i = S_i \qquad QHR \leq Q0 \qquad (7)$$

$$SQ_i = S_i - (SCAP)(QHR-Q0)/(QCAP-Q0) \qquad Q0 < QHR \leq QCAP \qquad (8)$$

$$SQ_i = SCAP - (SCAP-SJAM)(QHR-QCAP)/(QJAM-QCAP) \qquad QCAP < QHR \leq QJAM \qquad (9)$$

where

S_i = free speed of vehicle type i in km/hr,
SQ_i = volume-influenced speed of vehicle type i in km/hr,
$SCAP$ = speed at practical capacity in km/hr,
$SJAM$ = minimum jam speed in km/hr,
QHR = hourly traffic volume in passenger car equivalents (PCE)/hr,
$Q0$ = base volume level in PCE/hr,
$QCAP$ = practical capacity in PCE/hr, and
$QJAM$ = maximum volume in PCE/hr.

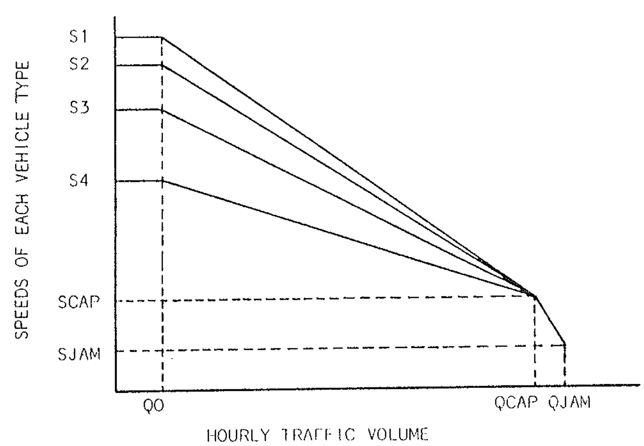

FIGURE 3 Three-zone speed-volume model.

It is recommended that *SCAP* be quantified as the 15th percentile speed of the slowest vehicle (*20*). Because speeds generally are normally distributed, this value can be approximated by the mean-minus-one standard deviation. Values for the coefficient of variation of vehicle speeds (defined as the mean divided by the standard deviation) that are based on New Zealand studies are given in another study (*24*). The speed at capacity is therefore calculated as follows:

$$SCAP = \min[S_i(1 - COV_i)] \quad (10)$$

where

COV_i = the coefficient of variation of speeds for type *i* vehicles.

The recommended values for *Q0*, *QCAP*, *QJAM*, and *SJAM* are given in the following table.

Parameter	*Value*	*Units*
Q0	100	PCE/hr
QMAX	2,500	PCE/hr
QJAM	3,125	PCE/hr
SJAM	8	km/hr

These values are expressed in terms of PCE/hr; it is therefore necessary to establish equivalency factors for the other vehicle types. A range of values from different studies is provided elsewhere (*20*). It must be emphasized, however, that because this model is not based on empirical observations, the values in the preceding table are only estimates.

In order to use the speed-volume modeling technique outlined earlier, it is necessary to know the hourly distribution of traffic volume. It has been found that for all rural roads in New Zealand, the hourly distribution is bimodal of the form shown in Figure 4 (*25, 26*). A typical distribution was selected that was based on a consideration of distributions from 11 sites (*1*). For modeling purposes it was assumed that there were five significant ranges of volume levels. These ranges and the number of hours that they can be anticipated to occur each year are given in the following table.

Volumes as Percentage of AADT		
Range	*Mean*	*Number of Hrs/Yr*
0-1	0.50	2,294
1-5	2.58	2,532
5-9	6.87	3,524
9-11	9.80	355
>11	11.66	51

The NZVOC2 model therefore divides an estimate of the annual average daily traffic (AADT) into five representative volumes, establishes the appropriate speed for each volume on the basis of the volume and mean free speed, calculates the vehicle operating costs, and then aggregates these costs to obtain the total annual cost.

EXAMPLES OF NZVOC2 MODEL PREDICTIONS

The NZVOC2 model can be used to provide standard tables of vehicle operating costs or to analyze individual segments of roads. The model provides two types of standard tabulated cost estimates: running costs and roughness-related costs. The former are composed of the costs that result from fuel, tires, oil, repairs, labor, and that portion of depreciation that results from

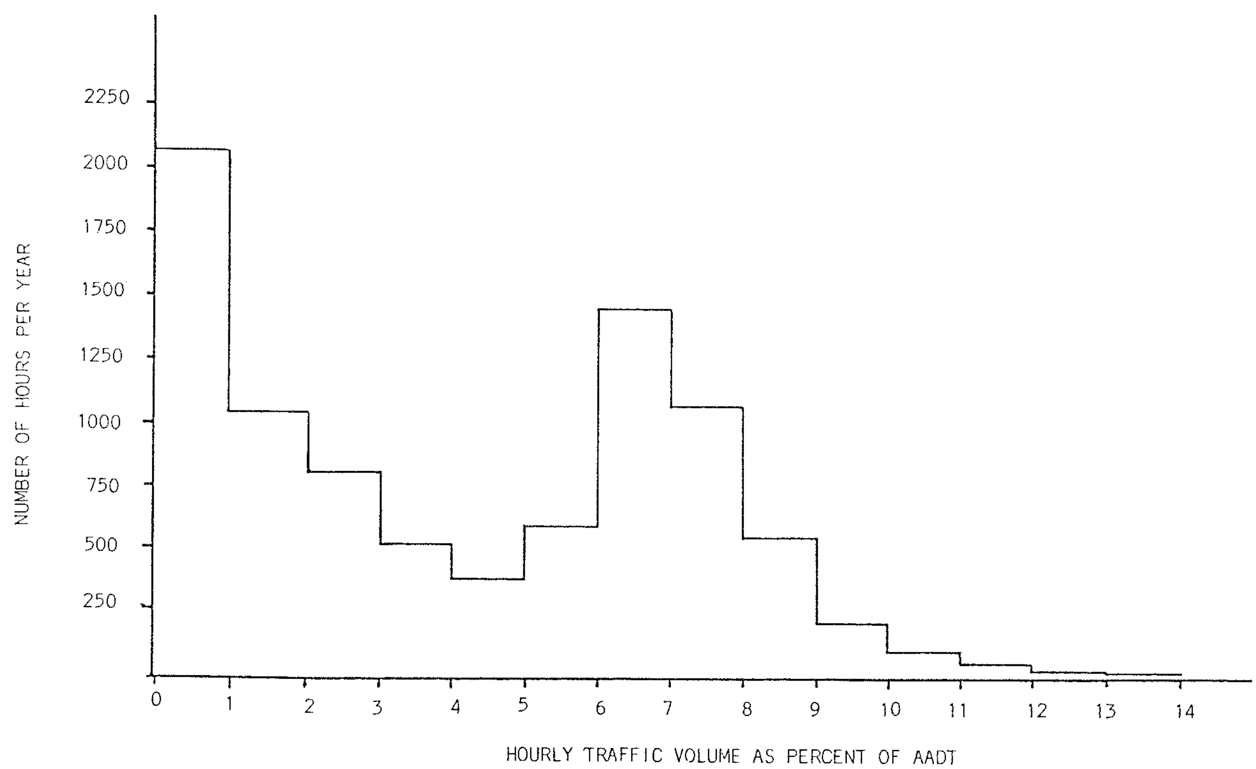

FIGURE 4 Hourly distribution of traffic volume.

vehicle use. The latter constitute the additional costs that result from changing roughness levels. Each of these costs is discussed individually.

Five significant independent variables were used in the calculation of the running costs. In increasing order of importance they are altitude, curvature, roughness, gradient, and speed.

The altitude affects the operating costs through its effects on the mass density of air. It has a very minor impact that amounts to less than 1 percent over the entire range of altitudes in New Zealand. Because of this insensitivity, all costs were calculated using a value of 10 m above sea level for the altitude.

The main impact of curvature on operating costs is through its effect on vehicle speeds. In terms of a direct effect on operating costs, only commercial vehicle tire consumption is affected by curvature. The running costs were calculated using a constant value of 10 degrees/km for curvature.

Roughness is an important variable, as evidenced by its inclusion in each of the HDM-III model relationships. Unfortunately, the Australian Road Research Board (ARRB) was concentrating on urban traffic management; therefore, surface roughness is not incorporated in the ARRB fuel consumption model. In calculating the running costs, it was decided to eliminate the roughness effects by having all costs calculated at a roughness of zero National Association of Australian State Road Authorities (NAASRA) counts/km. Separate roughness costs could then be added to these base costs to establish the total vehicle operating costs. Because the HDM-III model uses the QI roughness measure, whereas in New Zealand roughness is measured using a NAASRA Meter, it was necessary to convert NAASRA counts to QI counts (*27*). This was done using the following approximate relationship (*17*).

$$QI = (46.7 + NAASRA)/2.761 \tag{11}$$

where

$NAASRA$ = roughness in NAASRA counts/km, and
QI = roughness in QI counts/km.

It must be stressed that this is a very approximate relationship based on estimates of the relationship between the NAASRA Meter and the TRRL Bump Integrator and the Bump Integrator and the QI measure. The Ministry of Works and Development undertook field studies to quantify a more exact relationship; however, there were errors in the data. It is hoped that these errors will be resolved and that a more accurate relationship will be available in the near future.

The additional roughness costs were established by first calculating a base operating cost at zero roughness. Because the effect of roughness on commercial vehicle fuel and tire costs is itself affected by vehicle speed, the base costs were calculated over a range of speeds. The additional costs of operating on roads with roughness levels above zero were determined by calculating the average operating costs at each roughness level and subtracting the base roughness cost. The additional roughness costs for each class of representative vehicle are depicted in Figure 5 (*4*).

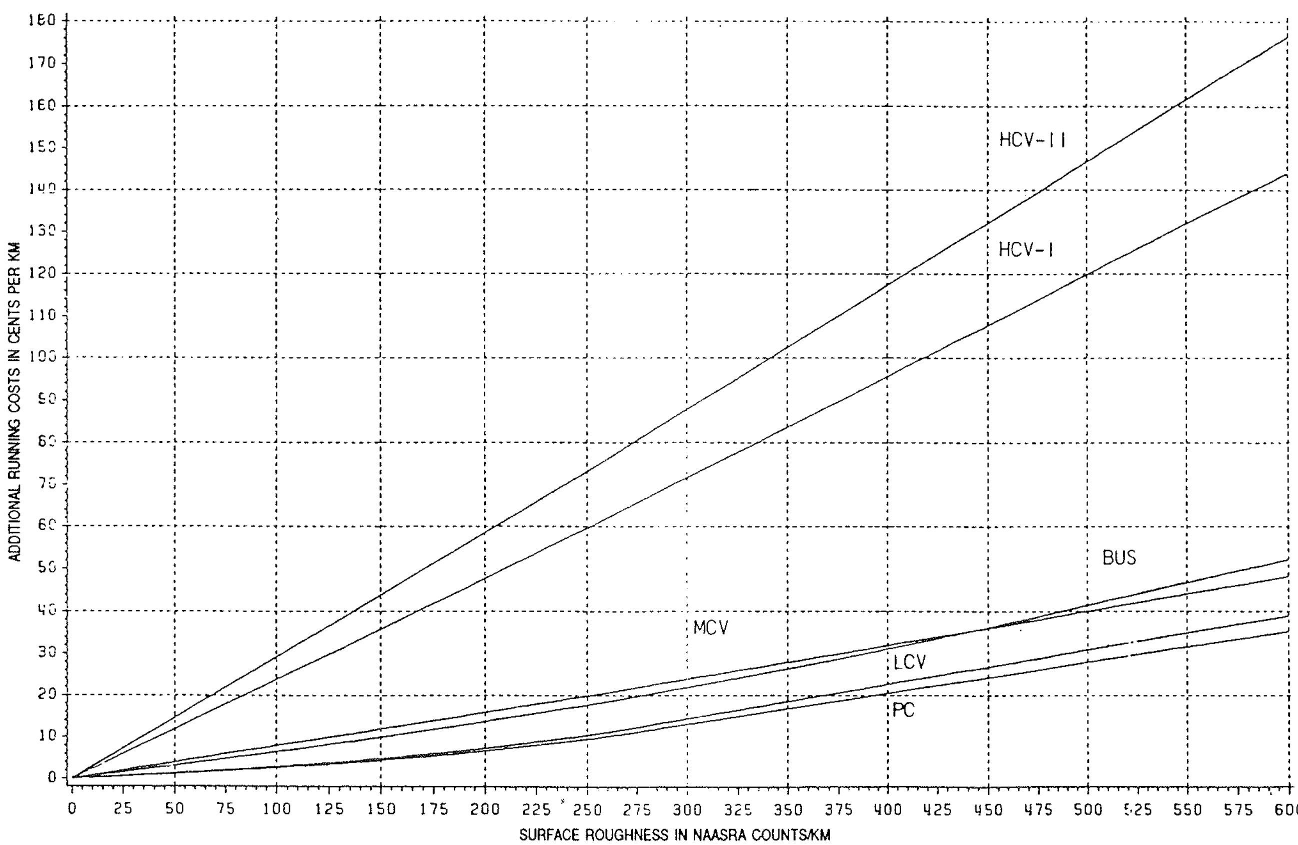

FIGURE 5 Additional running costs as a result of road roughness.

The running costs were calculated for a range of speeds from 10 to 120 km/hr operating on roads with gradients from -10 percent to +10 percent. As was discussed earlier, these costs were calculated using the following values for the other independent variables: altitude—10 m above sea level, curvature—10°/km, and roughness—zero NAASRA counts/km. In addition, the standardized speed distributions discussed earlier were used for each of the vehicle classes.

The NZVOC2 model predictions for rural passenger cars are depicted in Figure 6 (*4*). Because the ARRB model is linear in regard to gradient, the curves are fully symmetrical. The same does not apply to commercial vehicles, as shown in Figures 7 and 8, in which the running costs for the commercial vehicle reach a minimum at approximately -3 percent grade before increasing again (*4*).

The costs increase again because of the mechanistic commercial vehicle tire consumption model, which predicts that the tire consumption will be at a minimum when the sum of the forces that are opposing motion is at a minimum. Beyond this point, which occurs when the gravitational acceleration is equal to the rolling and aerodynamic resistances, the tire consumption can be anticipated to increase.

A complete set of operating cost tables and curves is presented in another publication (*4*). In addition to costs for individual vehicles, this publication contains cost estimates for standard mixes of traffic. This simplifies the manual appraisal process, because an analyst only has to refer to a single table or curve to estimate the costs. The NZVOC2 model is designed to prepare cost estimates for mixes of traffic; all that is required is the specification of the distribution of vehicle classes in the traffic stream.

The NZVOC2 model has not yet been used to independently evaluate single segments of highway, although it has been used in a research role to estimate the total road transport costs for New Zealand (*1*). It is anticipated that further developments of the model will be undertaken, specifically in the form of improved error reporting facilities, and that the model will be eventually released for use as an economic appraisal tool to supplement the manual method. Copies of the model that are suitable for running on an IBM personal computer are available by sending a floppy disk to The Technical Secretary, Administration Committee, National Roads Board, P.O. Box 12-041, Wellington North, New Zealand.

There will be a small charge to cover documentation and postage costs. Any requests for the Technical Recommendation for the Economic Appraisal of Roading Improvement Projects may also be sent to this address (*4*).

CONCLUSIONS

The New Zealand Vehicle Operation Model, NZVOC2, was developed specifically to meet New Zealand's requirements for an easy-to-use model that could be employed to estimate vehicle operating costs for use in highway economic appraisals.

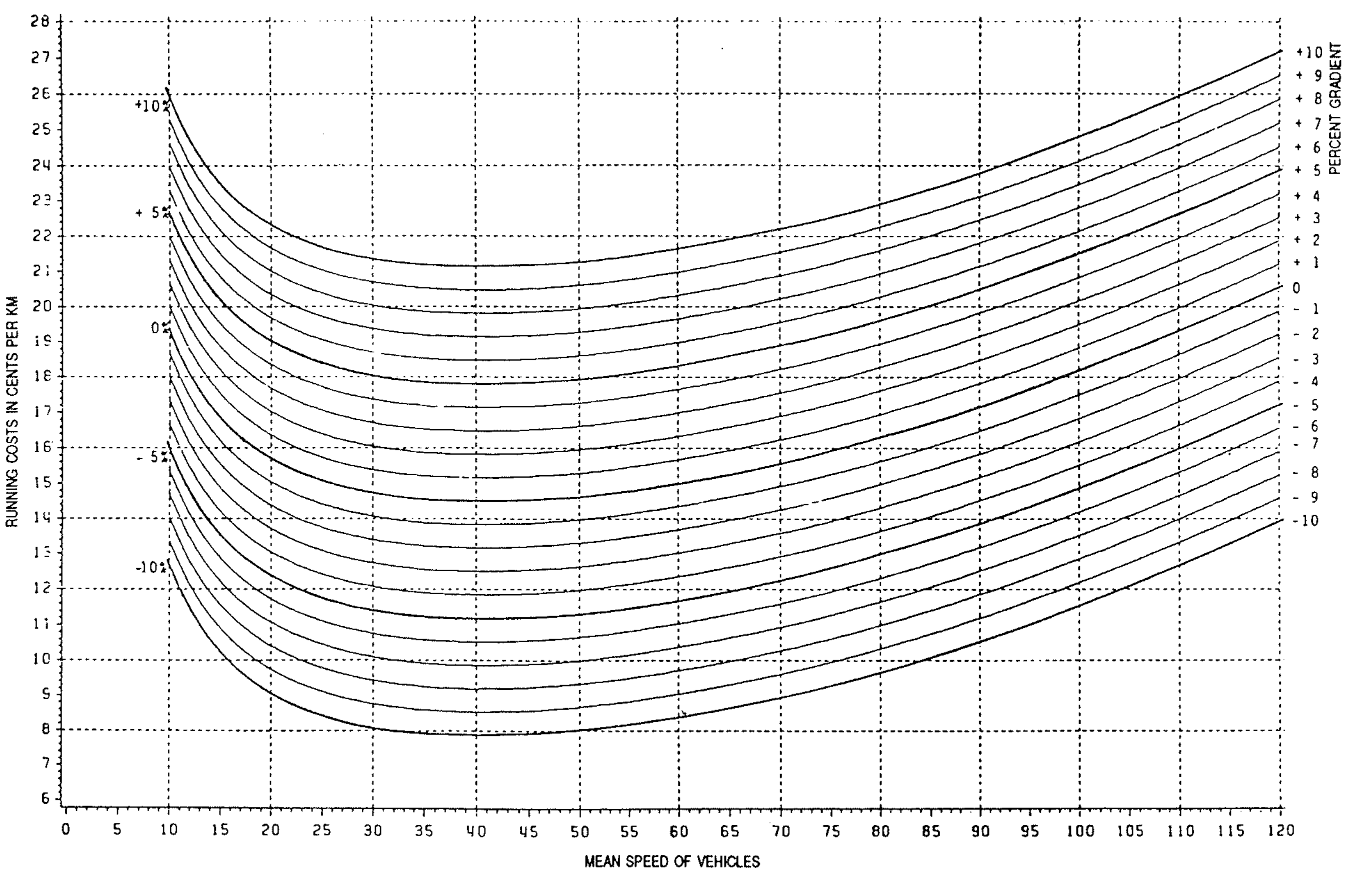

FIGURE 6 Rural passenger car running costs.

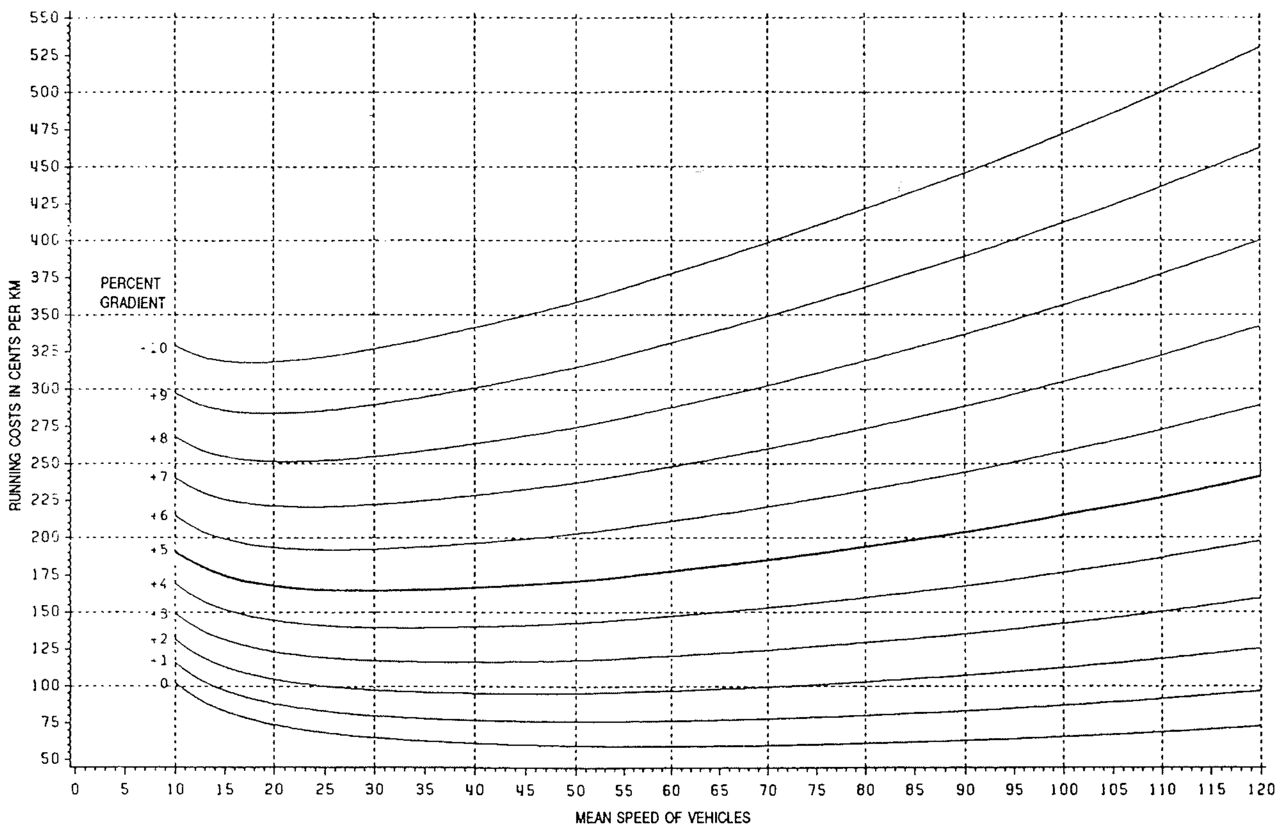

FIGURE 7 Heavy commercial vehicle (HCV-II) running costs on positive grades.

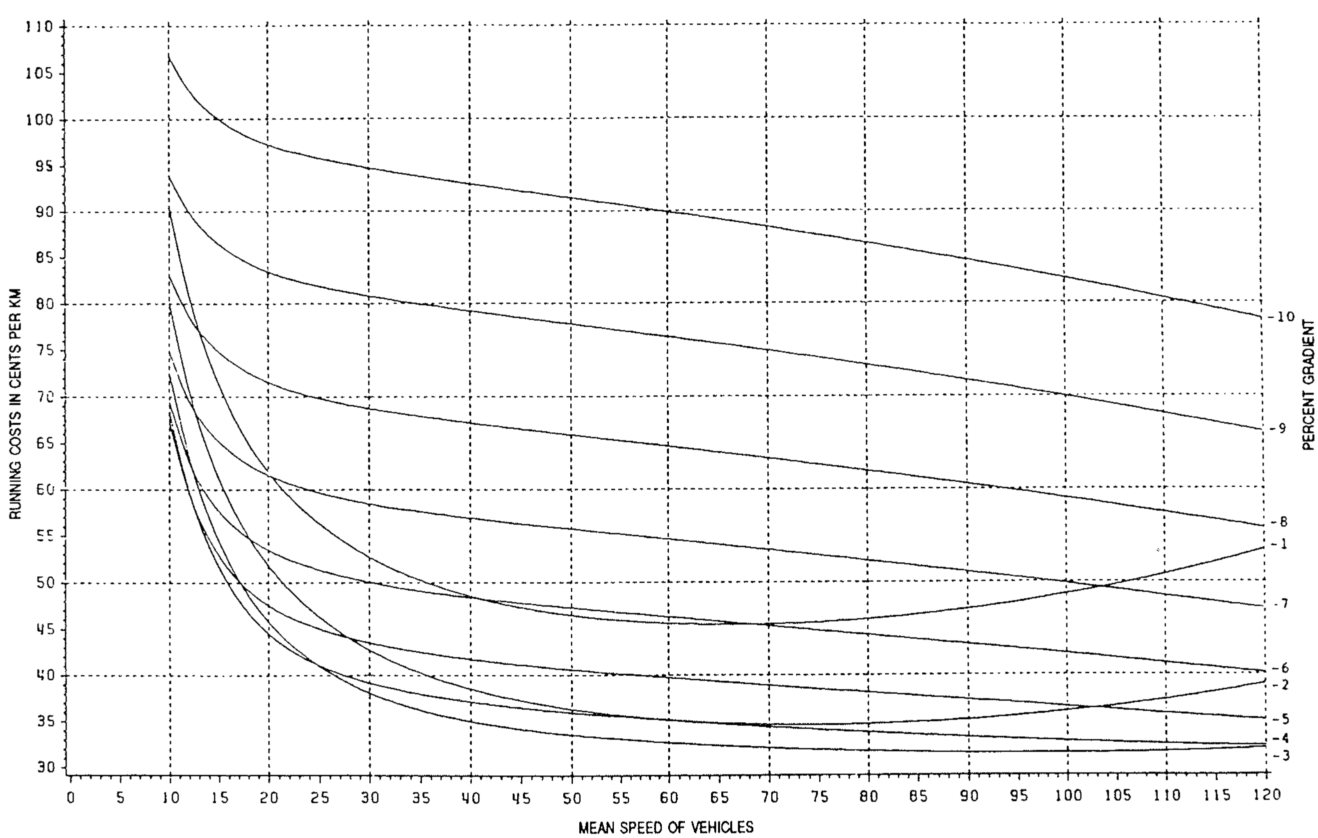

FIGURE 8 Heavy commercial vehicle (HCV-II) running costs on negative grades.

The relationships used in the model are primarily based on the World Bank's HDM-III model; passenger car and light commercial vehicle fuel consumption are predicted by use of Australian Road Research Board models. These overseas relationships were selected after their relevance to New Zealand vehicles and operating conditions was considered. Depreciation costs are based on the results of studies that were conducted within New Zealand and they successfully differentiate between the use- and age-related components of depreciation.

All countries that are considering the adoption of operating cost relationships that are not based on local research should undertake an evaluation similar to what has been done in New Zealand. Only then can it be ensured that the predictions are meaningful and relevant to the country in question. It is important that these predictions be accurate; given the nature of investments in low-volume roads, this is the only way to ensure the optimal allocation of funds.

ACKNOWLEDGMENTS

The author would like to acknowledge the National Roads Board of New Zealand and W. D. Scott Deloitte Ltd. for permission to present this paper. Appreciation is also extended to the Transportation Department of the World Bank, whose assistance made the development of the New Zealand Vehicle Operating Costs Model possible.

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Calibrating the Relationship Between Operating Costs of Buses and Road Roughness on Low-Volume Roads

P. C. CURTAYNE, A. T. VISSER, H. W. DU PLESSIS, AND R. HARRISON

Vehicle operating costs (VOCs) are a key element of user costs on low-volume roads and their prediction from highway design variables dominates economic evaluations of design and maintenance strategies. The results of a number of primary VOC studies are now available in which extensive experimental and survey data bases have been employed to determine a variety of vehicle cost-design relationships. Considerable efforts were made to ensure that these results would transfer well to other countries, although potential users are advised to ensure that they are appropriate for local conditions. Some of the difficulties encountered when trying to calibrate existing relationships to a regional environment are described. The discussion is based on a small-scale VOC study currently being conducted in South Africa to develop a set of cost-roughness relationships. Study resources are scarce and a priority program has been developed to identify those cost components that are most sensitive to roughness and that take more time to calibrate. General calibration issues are presented and the lack of guidelines is noted. Results on the calibration of bus VOCs are then discussed with an emphasis on a rolling resistance experiment to permit the mechanistic determination of fuel consumption and the prediction of tire lives, maintenance parts, and depreciation charges from survey data. Recommendations and general conclusions based on the experience of the study to date are then made to assist users who are considering the calibration of primary VOC relationships.

Relationships between vehicle operating costs (VOCs) and highway design variables were reported from the Caribbean, Brazil, and India at the Third International Conference on Low-Volume Roads in 1983 (*1-3*). Primary research on this scale is both expensive and time-consuming. Therefore, an attempt was made to develop relationships that would be applicable over a period of time and could be transferred efficiently to other environments. Many of the results obtained are now being employed in evaluation models such as the World Bank's HDM3 and the Transport and Road Research Laboratory's (TRRL's) RTIM2 (*4, 5*). The results are also being employed as the basis for smaller models that evaluate specific design and maintenance issues on low-volume roads, for example, the determination of grading frequencies in Bolivia and fuel consumption in Brazil (*6, 7*).

The users of the HDM3 and RTIM2 models and those interested in incorporating reported VOC results in other evaluation and management systems are advised to ensure that the relationships are appropriate to local conditions. The process by which this is achieved can be termed calibration. Potential users who are seeking guidance in this regard usually discover that little is available; guidance ranges from a chapter in the HDM3 manual to virtually nothing in the other studies.

In South Africa, VOC relationships are the main determinant of road user costs in pavement and maintenance management systems (PMS and MMS). In the development of the maintenance and design system (MDS) for unpaved roads, provisional cost-roughness relationships were derived from the Brazilian results after intercept values were adjusted to reflect local prices. These cost-roughness relationships for buses are depicted in Figure 1. The magnitude of the various cost components and their sensitivity to road roughness demonstrate the need to correctly address nonfuel items in VOC relationships.

The evidence from the primary studies indicated that VOCs were sensitive to roughness but that this relationship varied between the different components. Most important, significant variations existed in the cost magnitudes and differentials reported by the various primary studies. These variations highlighted the need to develop a series of cost-roughness relationships specifically for representative vehicles in southern Africa.

It was decided that these relationships would be most effectively determined by conducting a small-scale VOC study to evaluate the available cost-roughness relationships and calibrate them for local conditions. A detailed understanding was first developed in the study of the various primary studies; operating cost data were then collected for a variety of vehicle types. Bus data were the first set to be assembled and most of the discussion is devoted to the early efforts to calibrate existing bus relationships and to develop new ones when necessary. Lessons learned, however, are thought to apply to other vehicle classes.

Calibration issues are considered in the following section, which is followed by three sections that detail specific examples of different forms of calibration work undertaken to determine local bus operating costs. In the first of these sections the results are reported of an experiment to determine rolling resistance coefficients to predict fuel consumption from a mechanistic-kinematic model. Preliminary results are provided in the second section of tire consumption and maintenance costs. Depreciation charges obtained from a survey of bus operations are discussed in the third section. General conclusions and recommendations are then made in the final section.

CALIBRATION ISSUES

Calibration activities can be based on both primary and secondary data sources. Primary data are composed of direct

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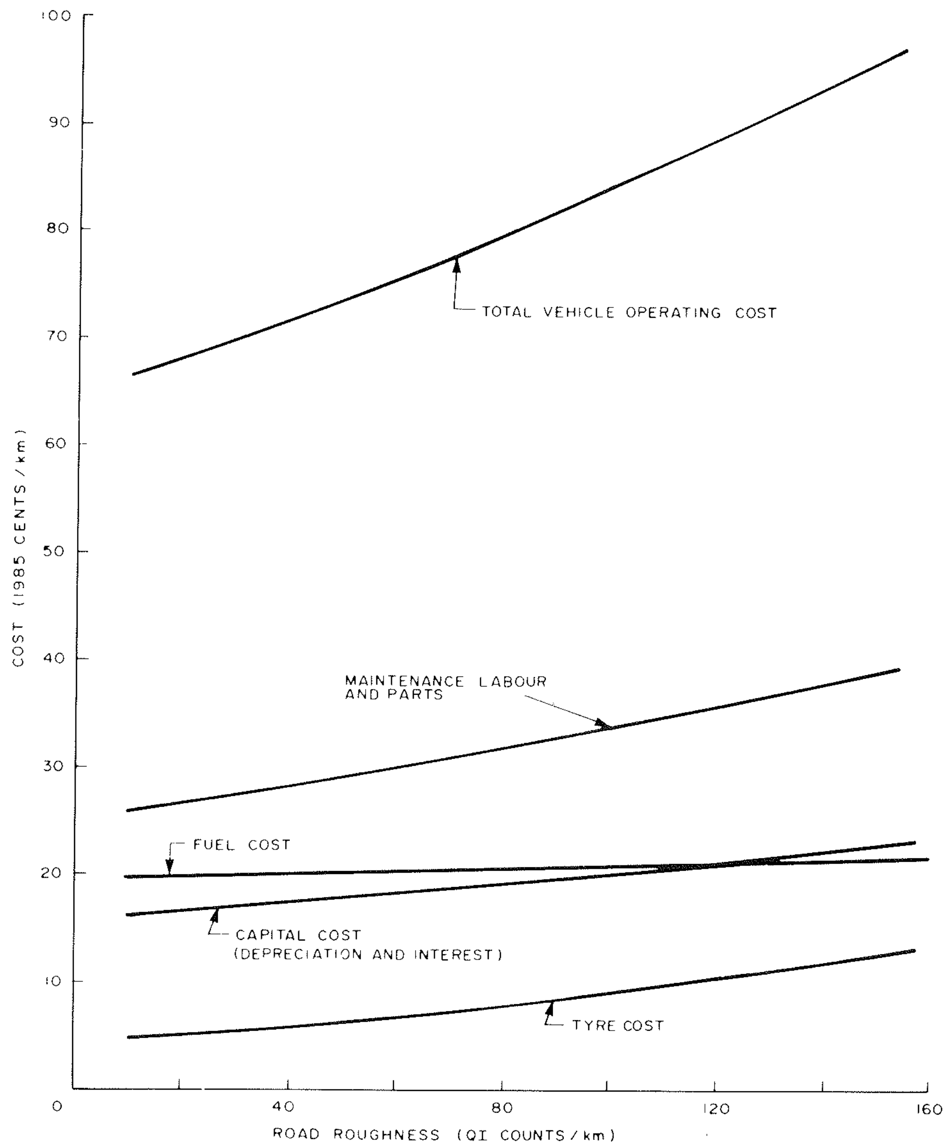

FIGURE 1 1985 vehicle operating costs for a bus on rolling terrain.

comparisons between costs and highway characteristics, whereas secondary data provide information about the economic or operating conditions of a region. The latter information can be used to adjust the results from primary studies that were conducted elsewhere.

The calibration of VOC results ranges from the selection of available relationships on the basis of few data to the estimation of local relationships using specially collected data and model forms reported in the various major, primary studies. The activities that have been identified in the small-scale South African VOC study are classified in Table 1. The impact of increasing calibration resources on data type and quality is shown in this table. Some users have only secondary data on which to base their choice and unless this information is good, it may be purely fortuitous if the relationships the users develop match local conditions. In cases in which good secondary data are available, mechanistic models can be modified with local inputs and empirical relationships can be selected on intercept values and slope differentials. Calibration work can be based on a combination of secondary data and experimental or survey activities if adequate resources are available. Most of the activities detailed in Table 1 represent different combinations of expertise, cost, and complexity, and they need to be performed in a comprehensive calibration exercise.

The basis for any calibration work is a good understanding of the research reports that provide the cost relationships of interest. This covers a wide variety of items that range from the scope of the research (vehicle types, research methods, and study size) to the economic environment at the time of the research. Economic growth and free competition, or regulation, for transport services influence loads carried, utilization, and vehicle speed. Knowledge of these effects in the countries in which the primary studies were conducted is extremely valuable when calibrating primary relationships. Unfortunately, most study reports do not adequately describe such effects. However, the World Bank compendium of results contains much relevant material to assist calibration activities (*8*).

Once the various, available primary studies have been examined and their regional macroeconomic and technical features have been assimilated, a preliminary choice of equations

TABLE 1 CALIBRATION ACTIVITIES DICTATED BY AVAILABILITY OF RESOURCES AND DATA

Type of Data	Sources or Needs	Increasing Resources Required		
Secondary Available	Government publications Industry technical reports VOC research reports Manufacturer's literature Consultants reports Operator organizations Road condition inventories	Choose primary study results which correspond to local conditions	Choose data sets and vehicle types Calibrate mechanistic models for technology changes (e.g. BHP).	Examine intercept and slope values for vehicle types with available cost and road condition data
Survey Calibration	Small-scale research collecting cost data from companies operating over a good range of highway characteristics. Personnel with industry knowledge Rates survey Route classification	Confirm intercept values and assess slope magnitudes Determine vehicle utilization by age and road condition	Estimation of tire and depreciation costs Confirm slope values for total costs	Estimate new local relations, especially for maintenance costs Compare predicted VOCs with rates
Experimental Calibration	Small-scale research of vehicle performance and roadway characteristics Trained personnel Analytical capabilities	Speed calibration	Fuel consumption calibration	Estimate new relations, for example, for rolling resistance and road roughness.

can be made. It was found useful in the small-scale study to plot the selected relationships so that a basis for comparison with local data could be established. This point may appear rather obvious but the major primary studies report a variety of different variable forms such as roughness, parts costs, labor values, and so forth, that could complicate comparisons between studies.

The resulting predictions give intercept and slope values, although the latter are generally sought in modeling roughness effects. The magnitudes of costs reported in the primary studies cannot be taken to represent general cost levels, even in the countries in which the studies were performed. This is because the surveyed companies were chosen for their operations over various combinations of route characteristics instead of because they form a representative sample of vehicle operators. Users who require cost levels in addition to cost differentials are advised to conduct an extensive calibration of equations to suit local conditions. In general, users should not mix equations from the various studies, especially for the maintenance parts, labor, depreciation, and interest results in which each study's results represent a coherent price/wage trade-off. If resources do not permit much calibration, then that study closest to the local environment should be chosen and efforts should center on the critical cost components that were identified by secondary data as being sensitive.

Data should then be collected. In some models this must be accomplished by experimentation, such as rolling resistance, and in other cases by a survey of the costs of vehicle ownership. In the case of survey data, a single company that provides a similar service over a representative range of highway types is the ideal company to survey.

Analysis of the Brazilian and Indian VOC survey data demonstrated the importance of variances between companies and within a company in the estimation of survey data. The use of a single company that operates over a wide range of a relevant independent variable, in this case roughness, avoids the error component of data between companies. The resulting data also have many other advantages, as detailed by Chesher and Harrison (*9*).

It is prudent to check the performance of the equations over the full range of highway characteristics, as defined by local conditions. Most of the equations, whether they are mechanistic-kinematic or empirically estimated, are extrapolated for extreme highway design values. Although it is claimed that mechanistic models perform better on extrapolation than empirical relationships, potential users would be well advised to check predictions with local operating data and to calibrate when necessary, because operators will try to reduce costs in precisely those areas in which they are predicted to be great.

Finally, a survey of transport rates can provide valuable information on the predicted costs of aggregate because the average rates charged by operators should reflect their total operating costs in a competitive economy. At the very least this would serve as a consistency check on total predicted values, whereas rate differentials could be employed to select primary relationships when calibration resources are scarce. When regulation or monopoly conditions affect the supply of transport services, rates data must be carefully interpreted before they can be used.

FUEL CONSUMPTION AND ROLLING RESISTANCE

In all primary studies, except the Brazilian study, the so-called aggregate-empiric approach was used to relate fuel consumption to road characteristics. Fuel consumption was measured directly in a number of types of instrumented vehicles on a great number of roads that varied in geometric properties and roughness. The required relations were then obtained by regression analysis.

The relations obtained through the use of this approach are inflexible because they cannot readily be adapted to suit vehicles with different engines or operating speed cycles. A new, large experiment would be required to obtain such impacts. An alternative approach was used in Brazil to model fuel consumption mechanistically by accounting for engine and drive-train maps, wind resistance, and rolling resistance. Rolling resistance is a function of road roughness and the suspension characteristics of vehicles. This relationship was observed in a small experiment with Brazilian vehicles. Similar, limited work in other studies has produced divergent results. An extensive experiment to obtain the relationship appropriate for local buses with a degree of confidence was accordingly conducted.

Test Procedure

The coast-down technique was used to determine rolling resistance. This involved coasting a vehicle down in neutral from a relatively high speed, usually about 80 km/h, over a road section the grade and roughness characteristics of which were known. Information on time and distance was recorded with a data logger at 1-sec intervals during testing, and the deceleration was computed. A constant cold tire pressure was maintained, and tire pressures were also measured during testing. Air temperature and wind speed were monitored, and tests were only performed when wind speeds were less than 4 m/s.

Two 92-passenger buses were used in the tests, which were performed on paved and unpaved roads that covered a range of roughness from 27 QI to 214 QI (1500 to 11 800 BI mm/km). The buses were fitted with 1100 × 20 cross-ply tires that were typically used by local operators. In a separate exercise, two 14-ton trucks that used essentially the same chassis, suspension, and tires as the buses were tested on a range of paved road sections the roughness of which varied between 16 QI and 73 QI (900 to 4000 BI mm/km).

Results of Rolling Resistance

It was found that the three levels of load conditions that were used and the texture of the road surface were not significant in defining the rolling resistance of the trucks. However, it was found that tire pressure affected the coefficient; consequently, tire pressures were carefully controlled and monitored. The analyses of the trucks and buses were performed separately. It was found that the roughness coefficients in the regression models were not significantly different for the different vehicles; consequently, the data were pooled. The following regression model was developed (*t*-values are in brackets):

$$A = \underset{(17.6)}{0.199} + \underset{(16.6)}{0.000322}\, QI - \underset{(-9.9)}{0.000177}\, TYREP \qquad (1)$$

where

A = rolling resistance coefficient (N/kg),
QI = road roughness in quarter-car index (QI counts/km), and
$TYREP$ = tire pressure (kPa).

This model had an R^2 value of 0.56 and a standard error of estimate of 0.0164. Four hundred and four observations were made. Each observation represents a single run. All coefficients are significant at 0.01 percent. The data points, which were standardized to a typical operating tire pressure of 640 kPa, and the prediction model are shown in Figure 2. The unpaved road sections were generally not as uniform in their geometric and roughness characteristics as the paved sections and this is reflected by a larger variance.

The physical interpretation of Equation 1 is as follows:

- An increase in tire pressure results in a reduction of the rolling resistance coefficient, in accordance with the evidence of tire mechanics.
- A decrease in road roughness results in a decrease in the rolling resistance coefficient. The improvement of a rough, unpaved road (200 QI, 11 000 BI mm/km) to a newly constructed, paved road (30 QI, 1650 BI mm/km) results in a reduction of 40 percent in the rolling resistance coefficient.

The implications of a constant 80 km/h fuel consumption are shown in Table 2. At a 640-kPa tire pressure, the fuel consumption on very poor, unpaved roads is 23 percent higher than on good, paved roads. An increase in tire pressure of 100 kPa

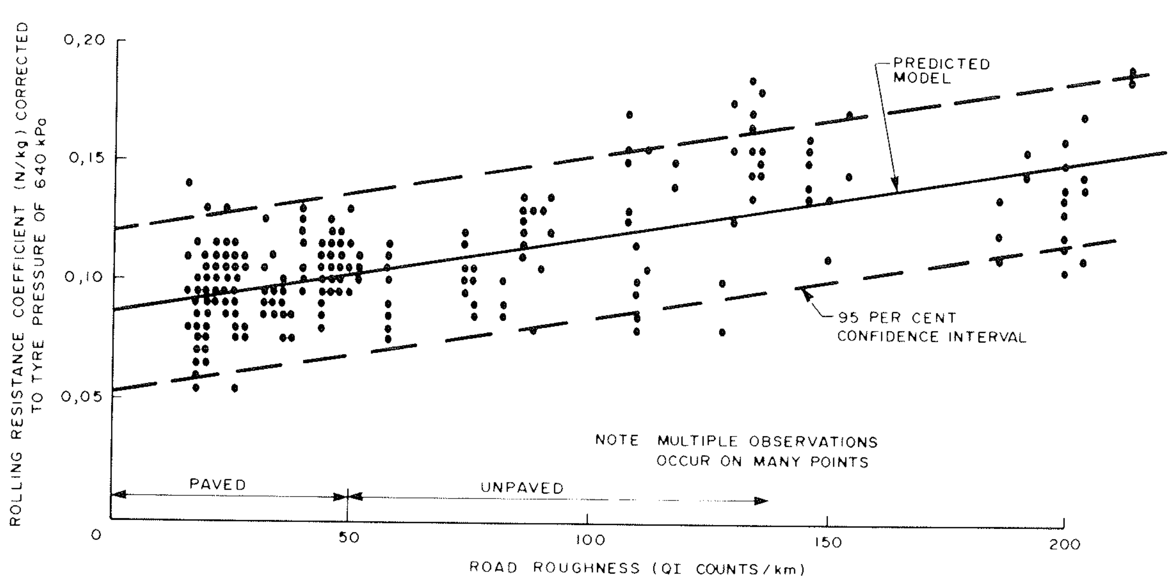

FIGURE 2 Comparison of measured rolling resistance, corrected to tire pressure of 640 kPa, with predicted model.

TABLE 2 ROLLING RESISTANCE AND ESTIMATED FUEL CONSUMPTION FOR 12 000 kg BUS AT 80 km/hr ON A LEVEL ROAD

Tire pressure (kPa)		Road Roughness (QI counts/km) Good paved (20)	Poor paved (80)	Very poor unpaved (200)
540	RR[a]	0.110	0.133	0.178
	FC[b]	30.7	33.0	37.4
640	RR[a]	0.093	0.116	0.161
	FC[b]	29.1	31.4	35.8

Note: 1 unit QI = 55 units BI.
[a]RR = predicted rolling resistance (N/kg).
[b]FC = fuel consumption (liters/100 km).

from 540 kPa to 640 kPa on a good, paved road results in a 5.2 percent reduction in fuel consumption. Although higher tire pressures reduce fuel consumption, a tire cost penalty is incurred when tire pressures are higher than the manufacturer's recommendations, and this could lead to premature failure.

Comparison With Other Studies

Relatively little work has been performed to relate rolling resistance to road roughness. The results of Watanatada and Bester are compared with examples of Equation 1 that demonstrate the effect of tire pressure in Figure 3 (*10, 11*). The results of the study by Bester show the same intercept value as this study but, even though a constant cold tire pressure was maintained in that study, the effect of road roughness was much greater. However, in view of the experimental scatter evident in Figure 2, the relatively small roughness range covered may have produced a spurious result. Watanatada also maintained a constant, but unknown, tire pressure. The coefficients of rolling resistance are generally higher than in this study but the roughness slope is a little less. Discrepancies could be the result of differences in vehicle characteristics or experimental error. In any event, the variability obtained in this study and the role of tire pressure indicate the care needed to define the scope of this type of experiment.

It should be noted that the fuel implications attributed to differences in rolling resistance apply to a constant speed of 80 km/h. In practice a driver would adjust the speed according to road conditions and the operating schedule. The driver could repeatedly change speeds on very rough roads in an attempt to avoid major deformations. Therefore, driver behavior should be studied to derive fuel consumption relations and compare them with aggregate-empiric studies.

The work described was performed on cross-ply tires; further work will be performed on radial-ply tires, which are gaining in popularity, even for use on unpaved roads. The effect of loose material on unpaved roads was not considered in this study, but it will be examined in future studies.

RESULTS OF THE USER SURVEY

A literature survey of available VOC relationships was conducted to group the selected equations by cost component and to plot their predictions. The economic and vehicle operating

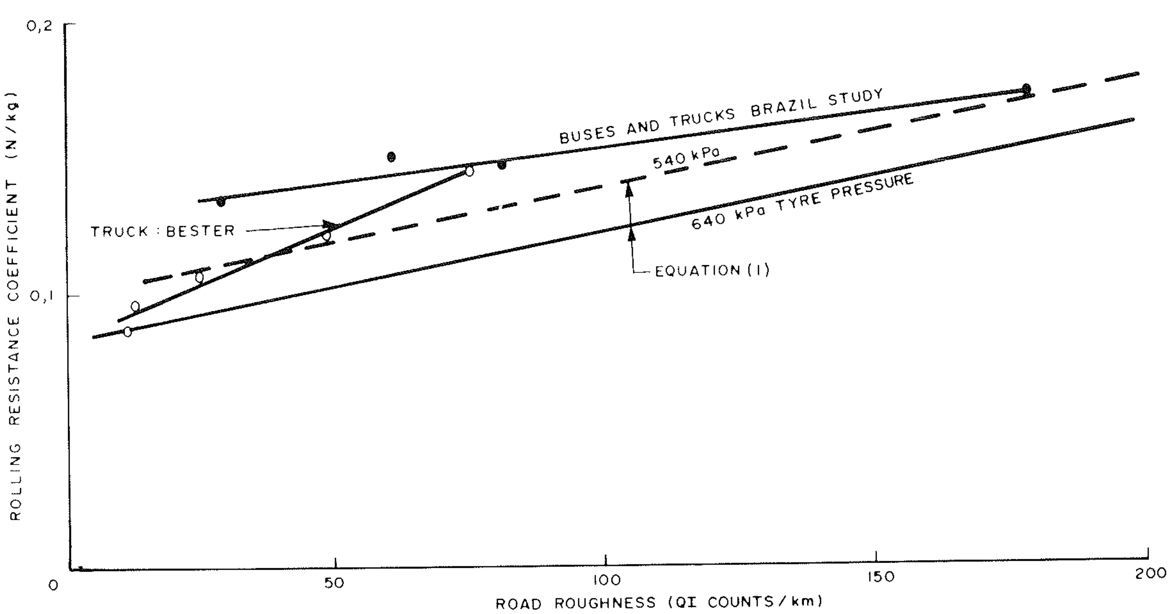

FIGURE 3 The effect of road roughness on rolling resistance coefficient for heavy vehicles.

conditions that prevailed in the relevant countries or regions were then noted. This provided valuable secondary information for the interpretation of predicted values.

For reasons that were detailed in a study by Chesher and Harrison, a single company that operated over a wide range of road roughness was sought to provide survey data (*9*). The following results are based on the data of such a company that operated 740 buses from nine depots with a combined annual fleet use of 50 million km. In order to provide the best range of road roughness values, operating cost data were taken from five of the nine depots over a complete calendar year, as recommended by Hide et al. (*12*). Traffic and workshop managers from the bus company were interviewed to ensure that vehicle operation and maintenance policies were fully understood and the cost data were correctly interpreted. Distinct highway characteristics were associated with the data of each of the five depots that were selected. It was believed that this would permit discriminations to be made between the major primary study results and would make the determination of equations for local conditions possible.

Roughness measurements were made with a response-type device, the Linear Displacement Integrator (LDI), and converted to QI units in counts/km. The network classification and use figures provided by the depots enabled sample measurements to be taken of the roughness of each of the five selected networks. Because individual route roughness values would affect the aggregate cost data in proportion to route length and use, it was decided to calculate a roughness average, weighted by fleet use, for each depot. The resulting weighted averages ranged from 42 to 120 QI (2300 to 6600 BI mm/km) and were used in the analyses of both tire and maintenance parts.

Tire Consumption

Each depot maintained a tire record system in the form of cards that detailed the history of each casing. Information was provided on the service lives of new and retreaded tires in kilometers traveled, and reasons were given for withdrawal from service. Monthly average tire costs were provided for both new and recapped tires. These costs and the service lives of the tires made it possible to calculate equivalent new tire life values. This calculation was first used in Kenya, and was subsequently adopted by all major studies. This is discussed in a study by Visser and Curtayne that can be found elsewhere in this publication. Equivalent new tire lives were derived from the following equation:

$$ENT = TK/(1 + NR/R) \tag{2}$$

where

ENT = equivalent new tire life (km/tire),
TK = total kilometrage per casing (km),
NR = number of retreads per casing, and
R = ratio of new tire price to retread price.

The new tire price was divided by the ENT values and the result was multiplied by the number of tires per vehicle to allow a comparison to be made with other studies.

Data were plotted and a simple log-linear relationship was determined from the various models that were fitted to the data. The relationship is as follows (t-values in brackets):

$$ENT = \underset{(6.6)}{152.1} - \underset{(4.7)}{25.5} \ln QI \tag{3}$$

$R^2 = 0.88$; standard error = 4.48.

where ENT = new tire life (10^3 km/tire).

This relationship is plotted in Figure 4, in which the relationships from Kenya, the Caribbean, India, and Brazil are also shown. Predictions of the number of tires per bus per 10 000 km are given in Table 3.

An examination of the data in Figure 4 and Table 3 shows a degree of similarity between the local bus tire predictions and those of the TRRL studies in Kenya and the Caribbean, whereas divergence is shown in the results of the Brazilian and Indian studies. Other independent variables, such as road geometry, vehicle age, and pavement width were significant in predicting tire usage in the Brazilian and Indian studies. However, this does not explain the significant differences in the service life versus roughness slopes. Because tire technologies are similar worldwide, other reasons must exist to explain the differences. Driver behavior, which is extremely difficult to quantify, could be one reason. Another reason could be the characteristics of the gravel materials.

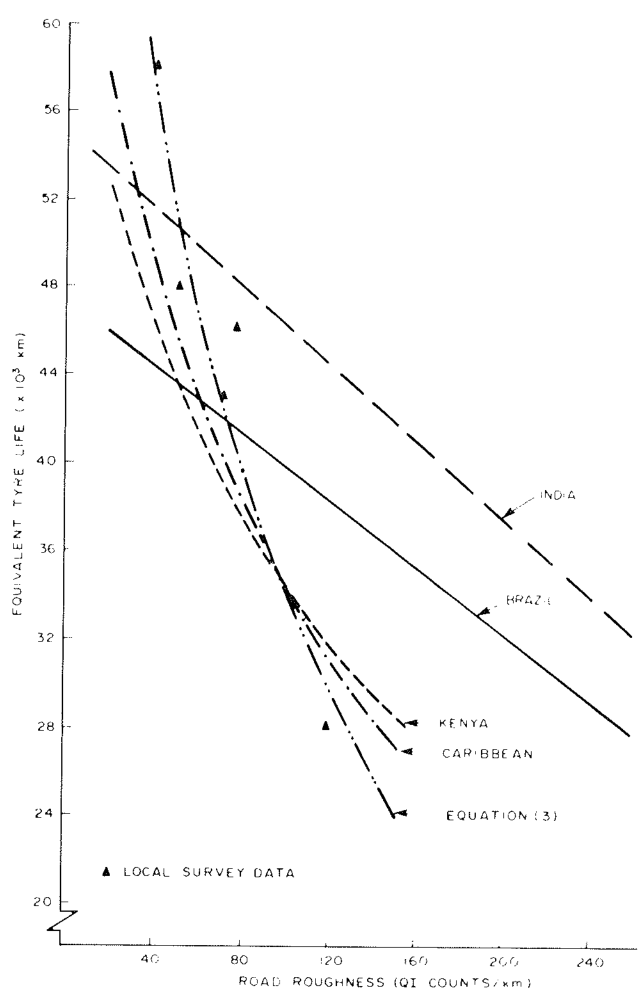

FIGURE 4 Predictions of equivalent new tire life for a medium-sized, 12-ton, two-axle bus.

TABLE 3 PREDICTION OF NUMBER OF TIRES PER BUS PER 10 000 km FROM VARIOUS STUDIES

Roughness (QI counts/km)	Kenya	Caribbean[a]	Brazil[b]	India[c]	Equation 3[d]
20	1.14	1.03	1.25	1.09	0.79
40	1.29	1.20	1.29	1.13	1.03
60	1.44	1.38	1.33	1.16	1.26
80	1.59	1.56	1.38	1.21	1.49
100	1.74	1.74	1.43	1.25	1.73
120	1.88	1.92	1.48	1.30	2.00
140	2.03	2.09	1.54	1.35	2.30

[a]Paved road operations only.
[b]Tire size 900 20; six tires per bus; rise plus fall 15 m/km; curvature 50 degrees/km.
[c]Vehicle age 250 000 km; six tires per bus; rise plus fall 15 m/km; curvature 50 degrees per km; pavement width 7.4 m.
[d]Six tires per bus.

It was reported at the depot with an average network roughness of 120 QI (6600 BI mm/km) that 45 percent of tire scrapes were caused by sharp, angular stones in the gravel wearing courses that penetrated the tire casings. It is evident from the available data that local gravel materials cause many premature failures and tread wear; further inquiries into this matter are scheduled. In addition, operators on routes of less than 35 QI (1925 BI mm/km) are reporting tire lives significantly higher than values predicted from the results of the primary VOC studies but similar to the extrapolated values of the local equation. Under these circumstances, the preliminary findings suggest that a local equation should be developed instead of a calibration of existing relationships.

Maintenance Costs

Maintenance expenditures are among the operating costs that are most likely to exhibit transferability problems. This is because maintenance expenditures are sensitive to price and wage levels and the trade-offs of depreciation and interest charges, all of which are linked to the size, strength, and structure of the local economy with the type of transport service offered by the operator. The bus company that provided the cost data is chiefly engaged in commuter work in which the reliability of service levels at peak periods is crucial to its success in business.

The company's policy was to set high levels of inspection and conduct preventive maintenance; these costs were balanced by keeping reserve vehicles to a minimum. Over 95 percent of the fleet was typically available for peak periods. During these times, the rest of the fleet received routine maintenance or was held in reserve. Engines, gearboxes, and differentials were changed at target lives of 300 000, 350 000, and 450 000 km, respectively, with a 10 percent variance to allow for local depot conditions. These expenditures were termed units and were treated as a separate cost item in the workshop accounts, which is a commonly adopted procedure in different parts of the world. Because the consumption of units was only partially related to road condition, it was not included in the preliminary analyses of maintenance costs.

The spares data, excluding units, were first examined by plotting costs against road roughness. Although a satisfactory linear relationship was developed, the results of all major VOC primary studies emphasized the importance of vehicle kilometer age in the estimation of maintenance parts costs. The local bus data were therefore assigned kilometer ages and the parts costs were normalized by dividing maintenance costs by new vehicle prices for the various makes and models supplied by company records. The provision of data across highway types from a single company ensured that all purchasing policies in regard to vehicles and spares were identical, and thus prevented the variations in policy from being ascribed to road roughness.

Various models reported in the major primary VOC studies were estimated and the following equation, which is similar to the Brazilian bus maintenance parts equation, showed the best fit:

$$\begin{aligned} \ln(SP/VP) &= \underset{(-6.25)}{-0.7894} + \underset{(13.61)}{0.4153} \ln(QI) \\ &\quad + \underset{(40.75)}{0.6313} \ln(AGE) \\ &\quad R^2 = 0.82 \end{aligned} \tag{4}$$

where

SP = spare parts costs in South African (SA) rands/10^3 km (2.5 SA rands = 1 U.S. dollar),
VP = new vehicle price (in 10^5 SA rands),
AGE = bus age (in 10^3 km), and
QI = road roughness (counts/km).

A model form similar to that reported by the Kenyan study yielded an R^2 value of 0.71 when estimated with local data, compared to the value of 0.92 reported in the Kenyan study. Equation 4, for a vehicle age of 250 000 km, is shown in Figure 5, in which the normalized predictions from the Kenyan, Brazilian, and Indian studies are also plotted. The results are regarded as encouraging and further work is to be undertaken on the other maintenance costs and the estimation of labor costs. The evidence of the roughness effect in the local data appears to be in accordance with the slope of the Brazilian relationship, more severe than the Indian data, and significantly less severe than the Kenyan data. The trade-off between maintenance flows and depreciation costs is crucial to vehicle operations because these costs determine purchasing, maintenance, selling, and scrapping decisions, and form a significant portion of total vehicle operating costs.

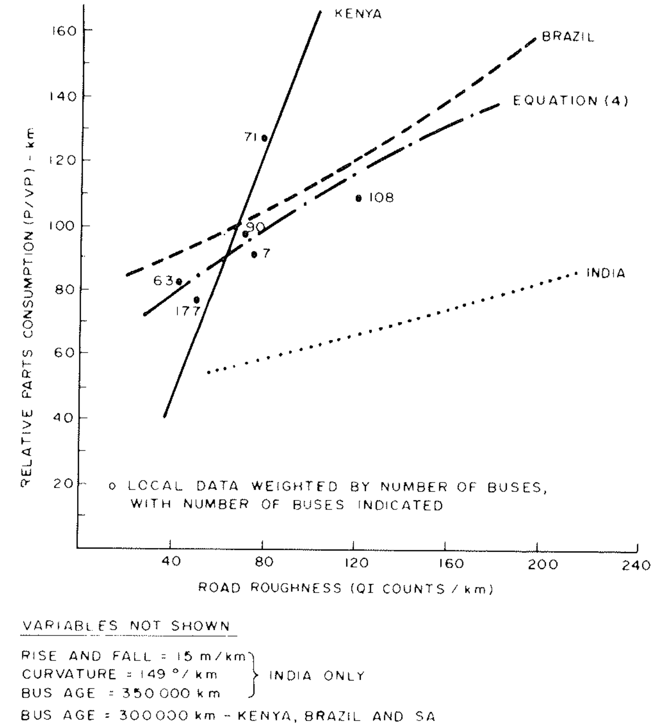

FIGURE 5 Relative parts consumption versus roughness values for buses.

VEHICLE DEPRECIATION

Depreciation costs, which reflect the change in capital value over time and use, can be substantial for vehicle owners, and can influence the costs of providing transport services. None of the primary studies reported relationships of vehicle value as a function of age (in km or calendar years) or of the route characteristics over which the vehicles traveled. All studies instead reported average vehicle age (in years) relationships that were generally obtained from national surveys of used vehicle prices. The effect of highway design characteristics appears to relate only indirectly to depreciation through the number of kilometers traveled. It is assumed that poorer roads lead to lower speeds and thus lower utilization although not necessarily proportionally. Therefore, fixed depreciation costs per time period can be placed on a per-unit time basis by changes in vehicle kilometers traveled.

The average vehicle life of 11 years that was reported by local survey companies was extended to 15 years if the vehicles were rebuilt. At a 15-yr service life, bus design and the availability of spare parts could make the safe and reliable provision of regular passenger service expensive. There is a point at which a case could be made to sell the bus when running costs exceed revenue for a particular service or scrapping the bus when depreciation and interest costs are zero. Chesher and Harrison have developed an optimal scrapping model that requires that vehicle life, s, satisfies the following:

$$\int_0^s \{m(s) - m(t)\}e^{-rt}\,dt = VP \tag{5}$$

where

$m(t)$ = the per year rate of running costs for a t-year-old vehicle,
r = per time period continuous discount rate, and
VP = new vehicle price (*8*).

It should be noted that Equation 5 is optimal only as long as vehicle use does not vary with vehicle age, a condition which can be satisfied by the provision of data by the local operator. All fleet vehicles are maintained to travel similar distances and must be able to operate over all road types in the depot network. If bus depreciation is calculated by use of vehicle-age relationships, the depreciation does not vary with highway design changes, because the use remains the same. If the main business of the bus companies is to provide commuting services with a relatively inelastic trip demand, then the vehicle-age method cannot produce depreciation differentials for inclusion in models that were designed to select highway maintenance strategies and options.

However, as detailed in a study by Chesher and Harrison, this is inconsistent with economic theory, which states that vehicles that require relatively high maintenance flows will command relatively low sale values (*8*). Improvements in highway conditions that lower running costs should lead to longer service lives or higher age values for the vehicles on the network. Imperfections in markets could make it difficult for prospective purchasers to know the use and maintenance patterns of the vehicles. Therefore, emphasis has traditionally been placed on the year of manufacture, which identifies the vehicle with a particular technological specification and a likely kilometer age. This practice may now be changing, however, because dealers in second-hand commercial vehicles in North America and Europe are offering 3-, 6-, and 12-month guarantees on parts and labor costs, which are influencing buyers in the manner predicted by economic theory. Given the discount rate, the new vehicle price, and the predictions of running costs over time, the optimal vehicle life can be determined using Equation 5. Terminal running costs can then be predicted to provide another means of determining total vehicle operating costs.

CONCLUSIONS AND RECOMMENDATIONS

The small-scale VOC study was designed to select a set of cost-roughness relationships that were reported in major primary VOC studies and to calibrate them to local conditions. Consequently, it was planned that very few equations would be estimated with local primary performance and cost data that were collected from experimental vehicles or operator records. A variety of calibration activities that were influenced by study resources and data availability was identified and detailed in Table 1. This made it possible to determine a program of priorities that reflected the sensitivity of the cost component in the VOC model and the modest resources at the team's disposal. It is likely that other potential VOC users will have insufficient resources to address all potential calibration activities and will need to focus on key components in a similar manner.

Vehicle operating costs are being developed for cars, buses, and trucks on low-volume roads. It was decided to first determine fuel consumption and tire and maintenance parts costs. The remaining costs, like depreciation, would then be addressed. Fuel consumption was to be predicted from a mechanistic model in which rolling resistance was the critical factor affected by road roughness. The other cost components would use calibrated aggregate-empiric relationships estimated from user survey data. The study is still in progress and most results are preliminary and tentative. In terms of the bus results presented in previous sections, the rolling resistance is considered to be well-defined for the front-engined chassis buses that are used in the study region. The results complement earlier research, for example, the research that was incorporated into the HDM3 model, and can be used with confidence for conventional buses and medium-sized trucks.

The tire analysis is proceeding well and seems to be in accordance with the Kenyan study slope for roughness of 40 to 100 QI (2200 to 5500 BI mm/km). Local predictions are more extreme than those of the major primary studies when a calibration curve is fitted to the data and extrapolated. As was previously noted, material properties on the rough, unpaved roads appear to be causing high failure rates, and this will be investigated. Local operators are reporting longer tire lives on good-quality roads that agree with the predictions of the calibrated curve.

The preliminary analysis of maintenance parts data is progressing slowly, as was expected given the problems of measuring this cost component. A cost-roughness relationship similar to that reported in the Brazilian study has been estimated and the model form reported in the Kenyan study also fits the data in a credible manner. No statistical basis currently exists for preferring a linear or log-linear form. The log-linear form was presented in this paper because it performed well when it was extrapolated. New discussions with bus company staff are proposed to check the data and the way data are being grouped. In addition, data from the remaining depots will be collected to determine if the discrimination can be improved.

Work in the area of depreciation has shown that cost differentials cannot be determined from the usual value-age method, because changes in speed do not affect use levels, at least in the short to medium term. In situations in which a road is improved, lower maintenance costs should alter the market value of the vehicle or its service life, and therefore its depreciation. The value-age method is typically insensitive to these effects and its use for local bus operations would not affect cost differentials. In an effort to maintain depreciation as a cost differential, an alternative economic model of optimal scrapping will be tested.

A general assessment of the study is beneficial to those who are contemplating the use of the VOC relationships in the primary studies. The majority of the aggregate-empiric results do not transfer easily and convincingly to the local conditions encountered. The role of calibration in which either few or many data exist is relatively straightforward. Few data permit only the selection of a set of relationships, such as in the Kenyan study, whereas many data permit local relationships to be estimated using model forms that were reported in the primary studies. Because good data are expensive to collect, the challenge to the research community is to develop effective calibration procedures based on secondary data or limited amounts of good primary data.

Encouraging results have been obtained in this study from a survey of a single, large company that operated over a wide range of road roughness. An understanding of operating policy and data quality can be easily gained by visiting a few depots, and inter-company variances can be avoided. Furthermore, this type of survey does not require great resources and provides a data source that is more informative than secondary data. It is hoped that this source of data will provide a basis for accurate calibration to local conditions. Cost differentials from such a source are likely to be extremely helpful in adjusting primary results. In addition, rates surveys are extremely valuable in situations in which competition and little regulation exist. Rates differentials should broadly give the same slope as that derived from total vehicle operating costs.

Finally, the development of mechanistic-kinematic evaluations models, most recently by the World Bank, is in part an attempt to avoid the transferability problems inherent in regression equations that are estimated from aggregate data. The small-scale study adopted the mechanistic argument for fuel consumption but is predicting the other cost components from survey data. Relationships for tire life can be modeled mechanistically with experimental data that were collected specifically to determine such a model. This approach has much to recommend it, but it was beyond the scope of this study. Maintenance parts and labor costs, and their trade-off with depreciation and interest charges, are likely to prove highly resistant to a mechanistic approach. Economics, not technology, is the key factor in the prediction of these components. The local VOCs that are being developed are likely to be a mix of aggregate-empiric and mechanistic models. When they are ready, they will be compared with other evaluation models instead of primary study results. Calibration is proving to be a challenging task and is proving to be more difficult than was anticipated at the initiation of the research. One of the objectives of this research was preparation of calibration guidelines, but the results are tentative at this stage, and the objective will not be fulfilled until the program is completed.

ACKNOWLEDGMENT

This work is part of an ongoing project of the National Institute for Transport and Road Research and this paper is published with the permission of its Chief Director.

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The Operation of Logging Trucks on Steep, Low-Volume Roads

PAUL T. ANDERSON, MARVIN R. PYLES, AND JOHN SESSIONS

The selection of a maximum grade for a road standard is a complex decision that involves design, construction, maintenance, vehicle, amount of use, and cost considerations. In the western United States, steep terrain, high construction costs, and the need to maintain slope stability make ridgetop instead of sidehill road locations attractive. Therefore, the gradability of steep (greater than 16 percent), low-volume roads primarily used by logging trucks and assisting vehicles is of major concern. Gradability is strongly influenced by the coefficient of traction, the most important variable, and by apparent truck grade and turning resistance around curves. The effect of grade on truck speed is also economically important. Truck safety depends on the road surfacing material and truck design, especially the truck braking system. As grades steepen, the amount of energy that the engine and service brakes must dissipate increases, and the rise in brake temperature can become critical. The road surfacing material has a major effect on truck performance. Gradation, particle shape, and in-place density of aggregate surfacing materials strongly influence the gradability of steep roads. Crushed-rock aggregate is preferred because it develops the greatest coefficient of traction under wet-season conditions, although some native soils develop higher coefficients of traction under dry-season conditions. Aggregate strength apparently increases as the fine-grained particle content increases under optimal moisture conditions. Careful planning can identify the conditions under which steep roads are most economical. Management alternatives include the use of assisting vehicles with logging trucks, surface stabilization to avoid erosion and added maintenance costs, and control of the season of use.

The selection of the maximum grade for a road standard is a complex decision that involves design, construction, maintenance, vehicle, amount of use, and cost considerations. Concerns over vehicle capability and operating costs have historically resulted in limited maximum road grades. More recently, however, the rise in road construction costs resulting from the need to maintain slope stability in steep terrain has prompted a review of recommended maximum grades and encouraged construction of roads at grades steeper than past maximums.

In the western United States, a combination of steep topography and erosive or unstable soils influences the range of physical options available. For example, approximately one-fifth of the roads constructed between 1972 and 1982 in the Mapleton District, Siuslaw National Forest, in the Oregon Coast Range were steeper than a 15 percent grade (*1*). Adverse (uphill) grades as steep as 26 percent for loaded logging trucks without an assisting vehicle and 30 to 35 percent for assisted logging trucks have been reported (*2*).

Road designers in the Oregon Coast Range currently prefer ridgetop instead of sidehill road locations to reduce overall road length and to avoid the cost of hauling and disposing of large volumes of excavated material. As the subgrade width and percent side-slope increase, the volume of excavated material increases dramatically (Figure 1). The USDA Forest Service estimates that ridgetop roads in the Coast Range cost an average of $100,000/mi, whereas sidehill roads cost from $250,000 to $600,000/mi. The disadvantages of ridgetop roads

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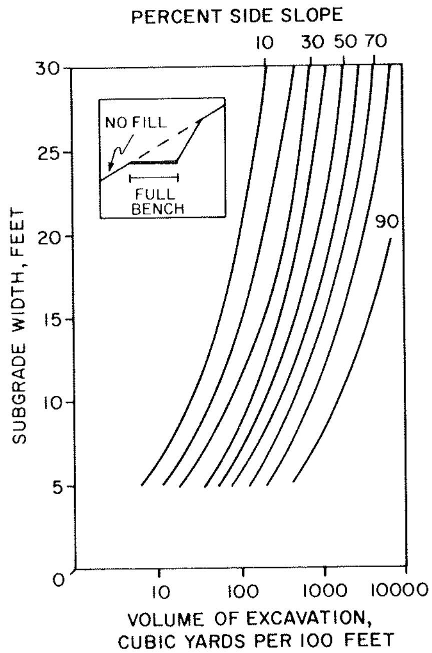


FIGURE 1 Excavation required for full-bench construction as a function of subgrade width and side slope.

are the need to use grades steeper than 20 percent (in some cases, steeper than 30 percent), higher direct operation costs (e.g., increased vehicle maintenance and the need for assisting vehicles), and increased potential for road surface erosion and therefore maintenance.

A summary is provided of the current knowledge of the design, construction, and maintenance of low-volume roads steeper than 16 percent that are primarily used by logging trucks in the western United States. In addition, the results are presented of an informal survey of engineers and road managers.

TRUCK OPERATION

Gradability

A major consideration in the operation of trucks on steep roads is vehicle gradability, which is defined here as the maximum grade that a vehicle or vehicle combination can climb or descend while maintaining adequate control.

A number of formulas for predicting logging truck gradability for straight road segments are available (*3-6*). These formulas, which are based on different sets of assumptions, describe gradability at different levels of detail. The most important variable is the coefficient of traction, which, unfortunately, has been quantified only generally. The wide variation in materials used for road surfacing and the degree of compaction apparently results in widely varying coefficients of traction. Caterpillar reports a 0.55 coefficient of traction for firm earth, Taborek reports 0.65 for dry earth (*7, 8*). This difference would change the maximum gradability of a loaded logging truck from approximately 23 to 28 percent.

In a study that related tire wear to slippage, Della-Moretta found that in cases in which tire slippage was less than 15 percent, the coefficients of traction for sand and gravel aggregate surfaces were less than 0.50 (*9*). If the coefficients of traction and truck geometry are known, gradability can then be calculated with the model presented by Sessions et al. (*6*). The developed slippage is related to the coefficient of traction in Table 1. Also shown is the gradability for a typical loaded logging truck in the western United States.

When traction is insufficient, an assisting vehicle is necessary to help a truck up the grade. For the purposes of this analysis, it is assumed that the vehicles can work together to produce the maximum thrust available from both vehicles. The basis of this assumption is that drive trains of the commonly used assisting vehicles (crawler tractors, rubber-tired skidders, and front-end loaders) can produce maximum thrust over the full range at a speed likely on a steep grade. For example, the gradability of a 76,000-lb loaded logging truck in combination with a 56,000-lb assisting vehicle would be approximately 38 percent if both the assisting vehicle and truck could develop a coefficient of traction of 0.55. The coefficient of traction of tracked vehicles is usually higher than that of wheeled vehicles on aggregate surfaces and unsurfaced roads (Figure 2).

TABLE 1 RELATIONSHIPS BETWEEN COEFFICIENT OF TRACTION, TIRE SLIPPAGE, AND GRADABILITY FOR A TYPICAL, LOADED LOGGING TRUCK IN THE WESTERN UNITED STATES

Coefficient of Traction	Slippage (%)	Gradability (%)
0.10	1.40	1.33
0.20	3.15	5.82
0.30	5.48	10.50
0.40	8.20	16.55
0.45	10.96	17.81
0.55	20.00	22.94

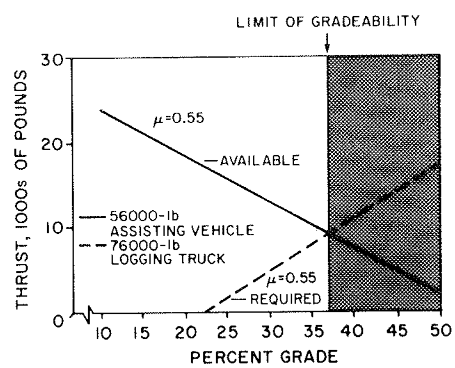


FIGURE 2 Thrust required by a loaded logging truck and that available from an assisting vehicle at various grades on a high-quality aggregate surface; μ = coefficient of traction. Shaded area indicates insufficient traction for the vehicle combination.

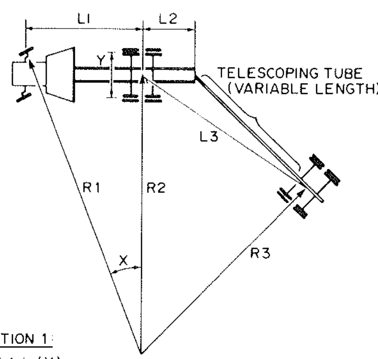
Gradability Around Horizontal Curves

The gradability of loaded logging trucks around horizontal curves is lower than that on straight road segments because of the additional resistance forces that arise from (a) off-tracking of the trailer, (b) the need to force the tandem axles around the curve, and (c) the inability of the tractor to apply thrust to the center of mass of the trailer along the direction of travel. Off-tracking is the difference in the path of the inside front wheel relative to that of the inside rear wheel as a vehicle or combination vehicle negotiates a curve (*10*). In addition, the available coefficient of friction to develop traction may have to provide lateral resistance to oppose centrifugal force on curves and thrust for forward movement.

Grade Resistance on Curves

As a loaded logging truck travels around a curve on a grade, the grade along the center line of the truck and trailer is different from the center line grade of the road, because the tractor and trailer tandems do not follow the path of the steering wheels around the curve (*3*). The trailer tandems off-track from the tractor tandems. The amount of off-tracking depends on vehicle geometry, curve radius, and curve length. Off-tracking causes the vehicle's center of mass to travel along a grade that differs from the center line grade of the road. The apparent grade that the tractor and trailer have to overcome must be included in a rigorous calculation of gradability. That grade will be steepest when the tractor and trailer have reached an equilibrium geometry (maximum off-tracking, e.g., in a tight switchback curve) for the curve. However, the trailer tandems often do not develop maximum off-tracking but move into and out of the curve on a transition pattern. Computing gradability for the maximum off-tracking case provides a conservative upper limit of gradability around a curve.

Maximum off-tracking of a stinger-steered, tractor-trailer combination on a curve with no side-slope to the road surface can be calculated by Equation 1 in Figure 3 (*3*). Once the relative positions of the axles on the curve are known, the relative elevations of the tires can be calculated and gradability can be determined. A formula recently published by the USDA Forest Service can be used to estimate tire locations for situations in which the vehicle does not develop maximum off-tracking (*11*).



EQUATION 1:

$$R1 = L1/\sin(X)$$
$$R2 = L1/\tan(X) - Y/2$$
$$R3 = \frac{R2 \times \cos\left[\sin^{-1}\left\{(L3/R2) \times \cos\left[\tan^{-1}(L2/R2)\right]\right\} - \tan^{-1}(L2/R2)\right]}{\cos\left[\tan^{-1}(L2/R2)\right]}$$

R1 = radius to outside front wheel in ft,
L1 = tractor wheel base in ft,
X = cramp angle of outside front wheel in degrees,
R2 = radius to center of tractor axles in ft,
Y = tractor width in ft,
R3 = radius to center of trailer axles in ft,
L3 = distance between the centers of the tractor and trailer tandems in nonturning motion, and
L2 = length of stinger in ft.

FIGURE 3 Off-tracking of a loaded, stinger-steered logging truck.

Turning Resistance of Tandem Axle Sets

As a tandem axle is pulled around a curve, drag forces are created. Smith suggested that the drag (D, in lb) that results from the forces aligning the tandems can be calculated as follows:

$$D = (u\ Wt\ L)/(2\ Rc) \tag{2}$$

where

u = 0.2 (lb/lb), a coefficient of friction associated with tandem cornering,
Wt = normal force on the tandem in lbs,
L = tandem spacing in ft, and
Rc = radius of curvature for the tandems in ft (*12*).

Effect of Axle Differentials on Gradability

Logging trucks are equipped with torque-balancing differentials on the powered axles. The effective thrust that can be developed is twice the thrust that can be developed by the most lightly loaded side of the tandems. It is therefore important to consider the normal forces on the driving tires. Both the angle of pull of the trailer on the tractor and centrifugal force can affect the distribution of normal forces. Because the trailer does not follow directly behind the tractor when going around a curve, the resisting force exerted on the tractor by the trailer is not directly in line with the thrust provided by the tractor tandems. The angle through which the force acts is the angle between the tangent to the arc that the tractor tandems describe and the tangent to the arc that the center of mass of the trailer describes. The resisting forces can be broken down into tangential and perpendicular components relative to the center line of the tractor. The perpendicular component acts toward the center of the curve and creates a moment that unloads the outside set of wheels. Centrifugal force can counteract this effect, but it is small at the low speeds reached on steep roads.

Effective Coefficient of Traction

The thrust produced by a moving vehicle depends on the coefficient of traction that can be developed on the road surface. That thrust must equal the magnitude and direction of the vector sum of the resisting forces. Because a major resisting force is not parallel to the direction of travel, the resultant of the coefficient of traction that must be developed also is not parallel to the direction of travel. This means that the effective coefficient of traction that can be developed parallel to the direction of travel is reduced to prevent the truck from sliding off the curve.

For a given coefficient of traction, μ, the gradability of a loaded logging truck on a curve can be determined by including in the calculations the resisting forces that result from cornering and climbing straight road segments. The grade-climbing ability of loaded logging trucks drops sharply on curves with radii less than 100 ft (Figure 4). The straight-segment gradability for a coefficient of traction of 0.45 is 17.8 percent; for a 50-ft curve radius, the maximum gradability is 13.7 percent.

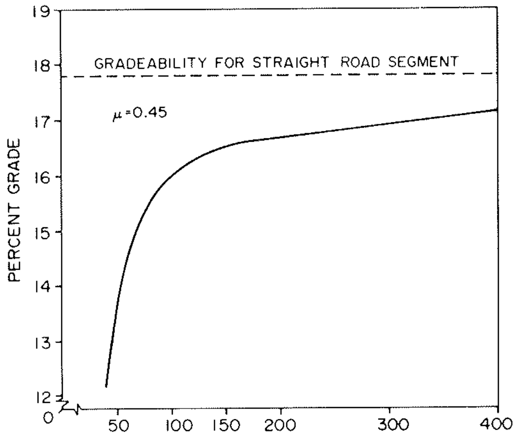


FIGURE 4 Grade-climbing ability around a curve for a typical loaded logging truck in the western United States; μ = coefficient of traction.

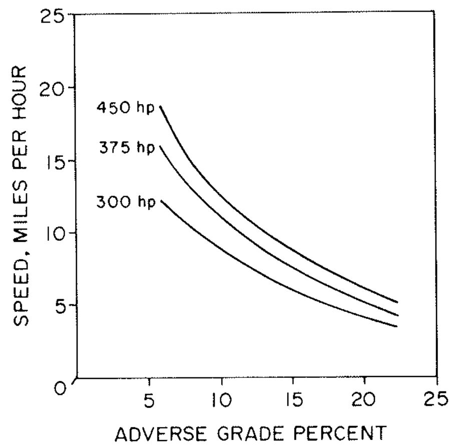


FIGURE 5 Horsepower-limited speed on adverse grades in which the effect of slip is taken into account (*4*).

Horsepower-Limited Speeds on Aggregate Surfaces

Although truck performance on steep, adverse grades is ultimately limited by available traction, the effect of the grade on truck speed is economically important because a slower speed increases hauling costs. On steep, adverse grades for which truck speed (V, in mph) is limited by available horsepower, not road alignment, McNally suggests the following formula:

$$V = (NHP \times E \times 375)/(RR + GR + AR) \tag{3}$$

where

NHP = net engine power in hp,
E = drive-train efficiency as a decimal,
RR = rolling resistance in lbs,
GR = grade resistance in lbs, and
AR = air resistance in lbs (*4*).

Air resistance can be ignored for speeds less than 30 mph. For steep grades that require the development of large thrusts, McNally's equation should be modified to include tire slippage as follows:

$$V = [NHP \times E \times 375 \times (1 - slip)]/(RR + GR + AR) \tag{4}$$

For instance, on adverse grades steeper than 15 percent, a loaded truck with 450 hp will be limited to a speed less than 9 mph (Figure 5) and may be even further constrained by gearing combinations (Table 1).

Superelevation

Superelevation exists any time a road has a cross-slope downward to the inside of a curve. The road surface may have been superelevated to produce a cross-slope for drainage or to counteract the vehicle side thrust that results from centrifugal force as a vehicle negotiates a curve. The superelevation required to counteract side thrust is a function of vehicle speed and road geometry. If the design speed for a given superelevation cannot be maintained around a curve, then the tractor could lean into the curve, which would reduce the normal force on the outside tractor tandems and, as a result, the available thrust due to the action of the drive axle differential.

Even the most powerful trucks commonly in service cannot travel fast enough on a steep, adverse grade to require significant superelevation. Log hauling contractors who were contacted in the informal survey reported that the use of superelevation on adverse grades steeper than 15 percent is not desirable.

Safety

Braking

The typical braking system on logging trucks in the western United States consists of an engine brake and air-operated service brakes. The Jacobs engine brake, a major improvement in logging truck brakes that was developed in the early 1960s, has become the industry standard. The braking force from the engine brake is applied to the road surface through the wheels of the tractor tandems. The maximum braking horsepower developed by an engine brake depends on the displacement, compression, and injection timing of the engine; the model of engine brake used; and engine speed. As road grades become steeper, the amount of energy the brakes must dissipate increases. For example, an 80,000-lb logging truck traveling at 10 mph while descending a 10 percent grade does not require the assistance of service brakes to maintain a constant velocity if the truck's engine brake and drive line friction can dissipate 285 hp. At a 15 percent grade, the service brakes must handle 9 percent of the braking load (dissipate 27 hp); at 20 percent grade they must handle 31 percent of the braking load (dissipate 129 hp).

Tests by Kenworth Motor Truck Company and Rockwell International Corporation on trucks with braking systems similar to those on logging trucks showed that the tractor-tandem service brakes often do relatively more work than the other service brakes (*13*). Because the engine brake also acts through the tractor tandems, the tractor tandems could lock up when trucks descend steep grades. One way to approach this problem is not to use the engine brake during steep descents; another way is to use only the trailer service brakes and the engine brake.

The percentage of the braking that each axle performs may not be proportionate to the static load it carries. Kenworth Motor Truck Company and Rockwell International Corporation performed tests of brake performance on tractor-trailer combinations (equivalent to loaded logging trucks) to determine the amount of braking performed by each axle (*13*). These tests showed that the tractor tandem does about 52 percent of the braking and carries 44 percent of the static load; the trailer tandem does about 41 percent of the braking and carries 44 percent of the static load; and the front axle does about 7 percent of the braking and carries about 12 percent of the static load. Because the tractor tandem brakes absorb a higher percentage of the energy used to stop the loaded vehicle, they would be expected to reach critical temperature first. Truck brakes normally operate at about 200°F and begin to fade substantially at about 650°F. Newcomb and Spurr developed the following equations to represent the temperature rise at the point of contact between the brake shoe and brake drum as the vehicle descends a grade and the brake is steadily applied to maintain a constant speed:

$$T = (2a^{0.5} \times t^{0.5} \times N)/(k \times 3.1416^{0.5}) \quad \text{when } L \geq 1.21 \qquad (6a)$$

$$T = (a \times N) \times (t + d/3a)/(k \times d) \quad \text{when } L < 1.21 \qquad (6b)$$

where

T = temperature rise in °C,
a = k/p in ft^2/sec,
k = thermal conductivity of brake drum in chu/ft/sec/°C,
p = density of brake drum in lb/ft^3,
t = time of brake application in sec,
N = energy input to brake drum in ft-lb/sec,
L = $[d/(a \times t)]^{0.5}$, and
d = thickness of brake drum in ft (*14*).

These equations do not account for cooling of the outer surface of the brake drum.

The amount of energy that must be dissipated can be readily calculated for a given speed and grade. The temperature rise in the wheel brakes can then be computed with Equation 6a or 6b. The results of these computations using the braking percentages from the Kenworth and Rockwell tests are shown in Figure 6 for three rates of energy dissipation of the service brakes (*13*). The distance that a loaded logging truck can travel at 10 mph before temperature rise becomes critical (650°F) in one set of tandems is shown in Figure 7. The distance was predicted by solving Newcomb and Spurr's equations for the time of braking application (*14*). The engine brake is assumed to dissipate 275 hp. When the engine brake and all the service brakes are being used, the critical brakes are in the tractor tandem axles. When the engine brake and only the trailer service brakes are being used, the critical brakes are the trailer brakes.

Other Safety Concerns

The basic design of the stinger-steered logging truck (Figure 3) presents some safety problems on steep grades. Under normal operation the only positive connections between the tractor and trailer are the logs. How tightly the logs fit into the bunks dictates how much assisting force can be applied toward pulling

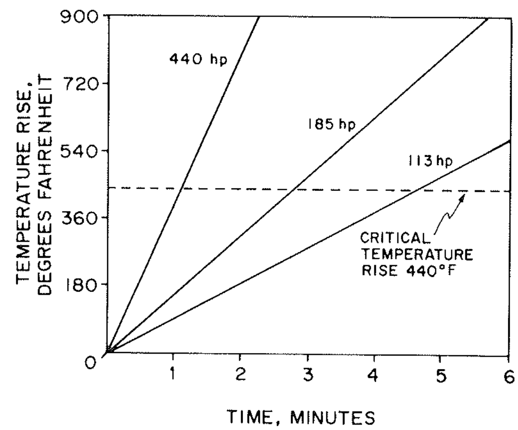


FIGURE 6 Temperature rise in the tractor tandem brakes as a function of energy dissipated by the total braking system.

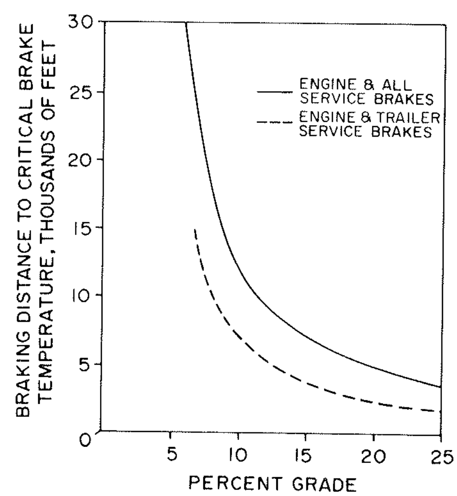


FIGURE 7 Braking distance traveled by a loaded logging truck at 10 mph before the rise in brake temperature becomes critical (650°F) in the tractor and trailer tandems. Engine brake is assumed to dissipate 275 hp.

the tractor or pushing the trailer or logs. Assisting vehicles have been known to push logs into the truck cab or pull the tractor out from under the load.

A road surfacing material that could lose surface traction rapidly also could be a safety hazard. For example, when asphalt concrete is clean and dry it may have a coefficient of traction of 0.8 or 0.9, but when it is contaminated with loose particles, it may have a coefficient of traction of only 0.3.

STEEP ROAD SURFACING

Surfacing Material

Crushed-rock aggregate is the major surfacing material used on all-weather, low-volume roads. The crushed rock should be thick enough to distribute surface wheel loads to the subgrade during critical wet-season months so that bearing capacity failure and associated rutting do not occur. On steep roads, subgrade support during the design life of the road is important; however, of equal and more immediate importance is the coefficient of traction of the road surface. Proper planning and management of steep roads require knowledge of the coefficients of traction of both aggregate- and soil-surfaced roads under various moisture conditions.

Gradation, particle shape, and in-place density of aggregate surfacing materials strongly influence gradability on steep roads. Crushed-rock aggregate is preferred for unpaved, all-weather roads because it develops the greatest coefficient of traction under wet-season road conditions. Under dry-season conditions, however, some native soils develop greater coefficients of traction. In fact, the highest coefficients of traction have been recorded on moist, fine-grained soils. The maximum, unassisted adverse climb observed was on a 26.4 percent grade in dry weather on a straight road segment composed of moist, nonplastic sandy silt. Published coefficients of traction for rubber tires on loose earth, firm earth, and clay loam range from 0.45 to 0.55, whereas the coefficient of traction for gravel is 0.36 (*7*).

Adverse gradability of loaded logging trucks could be limited to 16 percent when they are operating on pit-run aggregates that are loose and poorly graded (Unified Soil Classification System). If the gradation is improved, traction on pit-run aggregates could be improved; however, most engineers surveyed for this paper believe it is harder to produce a well-graded, 3- to 4-in minus aggregate than a well-graded, 1- to 1.5-in minus aggregate. A well-graded aggregate with a small maximum particle size appears to provide a higher coefficient of friction than large, open-graded aggregates. Trucks can often climb 18 to 20 percent grades without an assisting vehicle on well-graded, 1- to 3/4-in minus crushed rock.

A case history from the Sierra Nevada mountains in California illustrates some of the problems that can develop from using rock aggregate on steep road segments. Loaded logging trucks were unable to climb a 19.5 percent grade on a 185-ft-radius curve without an assisting vehicle after the road was surfaced with poorly graded 1-in minus crushed-rock aggregate and oiled. After clay from the cut bank was added to the oiled aggregate mixture on the steep section, which increased the fine-grained fraction of the mixture from 7 to 17 percent by weight (Figure 8), the trucks could climb the grade unassisted.

Casual field observations from the survey indicate that the preferred surfacing material for steep roads is crushed rock, but no quantitative guidelines were provided. Specific data relating coefficient of traction to aggregate gradation, particle shape, or in-place density are lacking. However, the available published data suggest the likely mechanism by which the traction limit of a rubber tire is exceeded on steep roads with aggregate surfaces. Although published coefficients of traction for rubber tires on a given surface vary, the coefficient of traction of a hard rock-like surface is consistently twice that of a rock aggregate surface (*7-9*). The characteristics of the individual particles within the aggregate surface probably explain much of this difference. If the coefficient of traction between rubber tires and the surface particles of an aggregate surface road approaches the published value between rubber and a rock surface, then traction failure on an aggregate surface must involve movement of the particles within that surface relative to one another. Particles probably both slide and roll within the surface; they also probably roll under the rubber tires. The coefficient of traction on a given aggregate surface is therefore limited by the stability of the individual particles that compose the surface layer. We have chosen to call this individual particle stability "matrix stability" to indicate all the particles of the surfacing material.

Matrix stability of an aggregate surface is believed to be related to the gradation of the aggregate and the plasticity of the fine-grained fraction. Ames and Vischer have reported some attributes of well-graded aggregate that correlate with acceptable performance relative to traction and maintenance (*15, 16*). Their measurements and observations of acceptable performance are interpreted to be an indication of matrix stability.

In a study of surface aggregate from roads with grades up to 16 percent, Vischer found that the oven-dried unconfined compressive strength of aggregates increased as a function of fine-grained particle content (Figure 9) (*16*). Tests were performed on samples compacted to maximum density by AASHTO T-99 and cured at 140°F for 48 hrs. Acceptable surface aggregate performance correlated with a minimum, oven-dried, unconfined compressive strength of 75 psi. When recommending gradations for road surfacing aggregate, Vischer limited the amount of allowable fines in an attempt to eliminate gradations that would lose strength under wet conditions (*16*). Although the unconfined compression test may not be the most appropriate test for evaluating road surfacing materials, Vischer's work indicates that it is a reasonable index of performance. Vischer's recommended surface aggregate gradations are shown in the following table (*16*).

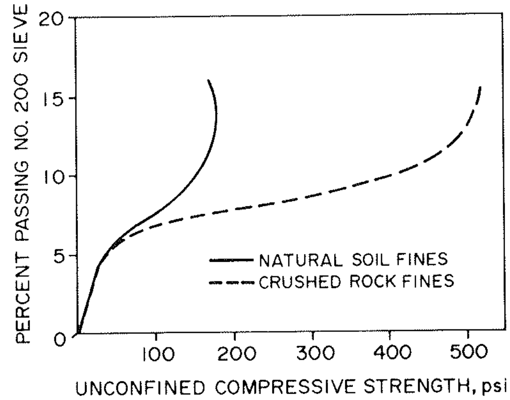


FIGURE 9 Unconfined compressive strength of aggregates as a function of fine-grained particle content (*16*).

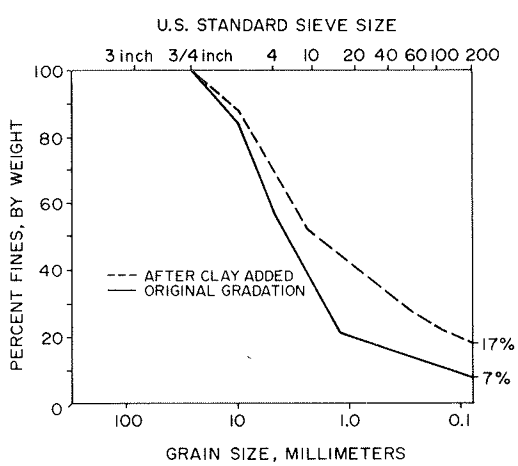


FIGURE 8 Effect on gradation of adding clay to an oiled, crushed-rock aggregate mixture on a steep road in the Sierra Nevada mountains.

U.S. Standard Sieve Designation	*Percent, by Weight, Passing Through Sieve*
1 in	100
1/2 in	68-80
No. 4	42-54
No. 10	26-38
No. 40	12-23
No. 200	7-12

Ames identified aggregate gradations that provided acceptable traction on grades from 14 to 20 percent (*15*). In order to assess aggregate strength, Ames recorded the length of sample that was self-supporting when extruded horizontally from a sample mold for aggregates compacted and cured at room temperature for 2 hrs. The tensile bending strength from Ames' results was computed, assuming the neutral axis of the sample was located at the center of the sample cross-section. Tensile strength increased as the percent, by weight, of the sample passing the No. 200 sieve increased (Figure 10).

Specifications relating to the plasticity index and amount of permissible fines do not apply to poorly graded aggregates as they do to well-graded aggregates (*15-17*). The void ratio is likely to be higher for poorly graded aggregates, which results in a lower strength, and, therefore, a lower matrix stability. Vischer recommended a narrow gradation band to ensure that only well-graded aggregates could meet specifications (*16*). If poorly graded aggregates are considered for use, their strength should be tested relative to available well-graded aggregates, because large differences in strength are likely to produce similar differences in the matrix stability, and, therefore, the coefficient of traction.

Both Vischer and Ames found that aggregate strength increased as fine-grained particle content increased (*15, 16*). It is currently inferred that the matrix stability and coefficient of traction are directly related to aggregate strength; however, further study is required to clarify this relationship.

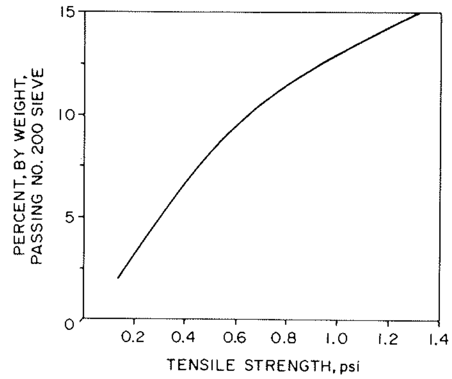


FIGURE 10 Tensile bending strength as a function of the fine-grained particle fraction (*15*).

Maintenance

Maintenance is often cited as one of the reasons steep roads are impractical, especially when maintenance problems are compounded by less than ideal weather and materials (*18*). As the road grade increases, both the road surface and ditch line require more maintenance.

In the Oregon Coast Range, because of unstable or erosive soils and high-intensity rainfall, most roads that are intended to be kept open must be surfaced whether or not they are intended for all-weather use. Maintaining these surfaces on steep roads is a problem. Graders must blade up the surface, reblend it, and spread it back out over the road. Some maintenance managers

TABLE 2 EFFECT OF ROAD GRADE ON DESIGN, CONSTRUCTION, MAINTENANCE, AND USE CONSIDERATIONS FOR STEEP, LOW-VOLUME ROADS

	As Road Grade Increases	
Consideration	Price per Unit	Quantity
Excavation	Stays roughly the same	Amount of excavated material decreases rapidly for ridgetop roads
End haul	Could increase or decrease	Would vary with excavation volume
Rocking costs	Increase slightly	Decrease if road length decreases sufficiently
Culverts	Stay the same	Increase
Maintenance		
Blading	Increases on roads steeper than 16 percent	Decreases; less length to maintain
Ditching	Increases	Decreases; fewer sidehill roads
Surface treatments to improve traction	Stay the same	Increase
Log haul		
Unassisted	Increases slightly	Stays the same
Assisted	Stays the same	Increases
Design and administration	Costs increase 20-40 percent	Stay the same
Clearing and grubbing	Decrease for ridgetop roads	Decrease for ridgetop roads

surveyed suggest that the minimum depth of rock aggregate that needs to be reblended is twice the diameter of the largest aggregate particle. Therefore, if the maximum particle size is large, more material must be bladed and reblended, which reduces the productivity of the grader and limits the grade on which it can effectively work. On steep, adverse grades, other maintenance managers simply blade to a lesser depth and deposit the oversized material along the road sides, which effectively changes the surface gradation. As road grades become steeper than 16 percent, some managers surveyed have reduced the maximum particle size specified in their aggregate mixes from 1.5 in to 1 in or even 0.75 in to facilitate grader maintenance. Although maintenance costs on these steep roads are higher, they apparently have not been high enough to outweigh the advantages associated with building steeper low-volume roads.

Roads constructed at grades steeper than 20 percent are generally intended for temporary use only. These roads are meant to provide access to a limited area during the dry season, serve their function, and be closed. Closure involves relatively maintenance-free out-sloping of the road section, removing stream-crossing culverts, and replacing cross-drain culverts with water bars or rolling grades. As a result, the general consensus among engineers surveyed is that such roads have not had extraordinary long-term maintenance problems.

COMPARISON OF STEEP, LOW-VOLUME ROAD OPTIONS

Road designers who are considering building steep, low-volume roads should take into account construction, hauling, and maintenance costs and intended use. All other things being equal, they will choose the alternative with the lowest cost. However, the effect of road grade on excavation, surfacing, drainage, hauling, and maintenance must be quantified for each situation. The sensitivity of these factors to road grade in the Oregon Coast Range, in which steep ridgetops are often preferred to sidehill locations, is shown in Table 2.

Steep grades have proved to be a viable alternative for low-volume roads in the mountainous western United States. Designers and managers have decided to use steep grades after considering the economics, environmental constraints, topography, and physical limits of equipment. However, they still need to better understand how rock gradation affects the surface aggregate strength and coefficient of traction; how to reliably estimate the erosion hazard from steep grades; and how to handle safety concerns about vehicle operation. Despite these gaps in the present knowledge, steep grades are being built, and will continue to be built. Only through careful consideration of each situation can reasonable and appropriate choices be made.

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Logging Truck Speeds on Curves and Favorable Grades of Single-Lane Roads

RONALD K. JACKSON AND JOHN SESSIONS

Speeds of loaded logging trucks traveling on single-lane forest roads were studied at three locations in western Oregon to evaluate truck performance on curves and favorable (downhill) grades. Uphill speeds for the returning, empty trucks were also studied. The independent variables of grade, curve radius, width, ditch depth, superelevation, sight distance, time of day, and maximum engine-braking horsepower were regressed against speed. The results were compared with those predicted by the Byrne, Nelson, and Googins (BNG) method and the Vehicle Operating Cost Model (VOCM). Grades above 11 percent strongly influenced speeds; less steep grades only slightly affected speeds. The BNG study and the VOCM predicted downhill speeds reasonably well for straight sections of road with favorable grades between approximately 11 and 16 percent. The BNG method and the VOCM overpredicted speeds for grades steeper than 16 percent. Because of the poor alignment of the roads studied, no conclusions could be reached for favorable grades less than 11 percent. Speeds recorded for curves did not relate to the assumption in the BNG study that horizontal sight distance controls speeds or to the assumption in the VOCM that available friction controls speeds. These findings may influence future road designs. Sight distance may be less important because drivers now use citizens' band radios to learn about road conditions ahead. Technological improvements may also account for differences reported here. Overall, the BNG method underestimated round-trip travel speeds by 16.7 percent when its predicted speeds were compared with those observed over a 7.71-mile portion of a logging road.

The design and construction of a low-volume logging road influence the overall cost. A design that shortens hauling time reduces hauling cost. Therefore, it is important for designers to accurately predict how truck travel times relate to changes in road geometry so that the most cost-effective roads can be built. How truck speeds relate to road design is assumed to depend either on driver behavior or truck performance.

If a minimum-bid price is to be established for a timber sale, the appraisal of log-hauling costs can directly influence that sale. In some areas, log hauling can account for approximately one-third of the total harvest cost (*1*). Few published studies that relate log-hauling costs to truck travel times have been conducted recently. Therefore, many USDA Forest Service appraisers still consult some version of the 1960 study by Byrne, Nelson, and Googins (BNG) to estimate those costs (*2*).

However, truck technology has improved so much during the past 27 years that speeds and travel times that were estimated for older models of logging trucks may not be accurate today. For instance, the Jacobs engine brake has increased the sustained braking ability of logging trucks on favorable (downhill) grades. In addition, better engines, transmissions, steering mechanisms, and the citizens' band (CB) radio have contributed to safer, faster, and more efficient log transportation.

In addition to the BNG study, vehicle simulation models have been developed by universities, truck manufacturers, and others, but these models have not yet gained wide acceptance for use on low-volume roads. In 1975 the Forest Service participated in the development of one such model, known as the Vehicle Operating Cost Model (VOCM) (*4*). The VOCM uses physical relationships and a set of driver behavior rules to determine truck speeds.

The results are presented of a study conducted at Oregon State University during the summer of 1985 (*3*). Three major objectives of this study were:

- To compare the actual speeds of modern logging trucks on curves and favorable grades with the speeds predicted by the BNG method and the VOCM,
- To determine how well the BNG method predicts overall trip speeds by comparing estimated speeds with actual ones over a longer portion of road, and
- To examine if extrapolation of the BNG data is appropriate for grades steeper than those on which the original study was based.

The study was limited to favorable grades because most log transportation from the forest to the mill is usually on these grades. As commonly defined by road engineers, a favorable grade implies that loaded trucks travel downhill, whereas empty trucks travel uphill. This convention is used throughout this paper.

BACKGROUND ON FACTORS THAT AFFECT TRUCK PERFORMANCE

Although the BNG study and the VOCM can both be used to analyze truck travel times, they differ in their basic assumptions. In the BNG study, it is assumed that either grade or alignment determines truck speed. Uphill truck speed on straight sections of road is limited by horsepower. Downhill speed on straight sections of road is limited by sustained braking capacity. On curves, truck speed is determined by the sight distance required to stop. In the VOCM, however, grade and alignment are considered simultaneously. The VOCM assumes that sliding friction, not sight distance, determines speed on curves.

Grade

On sections of logging roads in which curves do not appreciably affect driving behavior, grade is considered to be the primary

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factor that controls vehicle performance (*2, 4*). Vehicle speed on these sections is assumed to be constant. That is, the sum of the retarding forces acting on the vehicle is equal to the force component of the weight parallel to the road. The VOCM assumes that the driver selects a gear that allows the maximum sustainable speed within the capacity of the engine brake without the use of the service brakes. This maximum braking horsepower is assumed to be equal to 320 at 2100 rpm. It is assumed to be an exact balance between braking power required and braking power provided. The equation used by the VOCM is as follows:

$$V = (550 \times B) / (W \times G + W \times R) \tag{1}$$

where

V = velocity in feet per second,
B = available engine-braking power in horsepower,
W = vehicle weight in pounds,
G = road grade as a decimal fraction, and
R = rolling resistance as a decimal fraction.

The BNG study developed an empirical relationship that related the speeds of loaded and unloaded trucks to favorable grades. That relationship was based on a variety of truck sizes and road surfaces for favorable grades that ranged from 2 to 16 percent. Only one data point was at 16 percent; the rest were 12 percent or less. The relationship developed was as follows:

$$V = \frac{240}{3 + G} \tag{2}$$

where

V = velocity in miles per hour, and
G = road grade as a percent.

The empirical relationship given by BNG for truck travel times on favorable grades is shown in Figure 1.

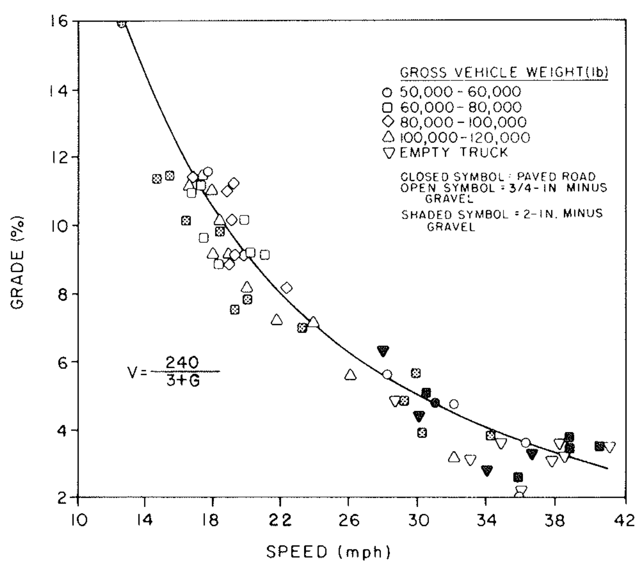

FIGURE 1 The influence of grade on the speeds of loaded and unloaded logging trucks traveling downhill on a straight section of road (*2*).

Alignment

Curves can affect vehicle speeds in two ways. They can limit sight distance so that speeds must be reduced and they create a centrifugal force that can make the truck slip or tip, depending on the available friction at the road surface. Vehicle speed on curves is ultimately limited by such slipping or tipping.

Sight Distance

Sight distance is the most commonly accepted factor controlling truck speeds on curves of single-lane logging roads (*2*). The driver will assumably maintain a speed that will permit a stop within the available distance between the two vehicles. The assumption used by BNG is that the driver anticipates meeting another vehicle traveling at the same speed. An alternative assumption often used in highway design is that the driver operates the vehicle at a speed that will permit stopping to avoid hitting an object on the road (*5*).

Sight distance is commonly defined as the line-of-sight distance between the driver's eye and an object on the road. The usual height of the driver's eye is assumed to be 3.5 ft and the object height is assumed to be 0.5 ft (*5*). Because this definition applies to average-sized vehicles, it may not adequately reflect the sight distance from the cab of a logging truck. BNG used an eye height of approximately 7.5 ft to calculate sight distance for various road widths. They also assumed a sighting point of 4.5 ft on an oncoming vehicle as the point that triggers the driver's braking reactions (Figure 2a). The hood of the approaching vehicle is assumed to be 4.5 ft above the road surface. Therefore, the average height of the tangent point on the backslope is (7.5 + 4.5)/2, or 6 ft, above the road surface (Figures 2a and b). The middle ordinate, M, is the horizontal distance from the center of the roadway to the point on the backslope that is just tangent to the line of sight between the driver's eye and the approaching vehicle (Figures 2b and c). The resulting relationships between curve radius and average truck speed, based on sight distance, are shown in Figure 3 (*2*).

Available Friction

The BNG study assumes that because sight distance restricts travel speed, it is less than a speed based on side-slipping friction. The VOCM, however, assumes that side-slipping friction determines travel speed. The authors of the VOCM base this assumption on the belief that the additional height of the driver's eye, the driver's experience and familiarity with the road, and the use of CB radios permit driving "beyond sight distance" (E. Sullivan, unpublished data).

Most typical log loads in western Oregon will probably not be affected by tipping because of the low coefficient of friction on gravel roads in that region. In addition, less friction is available to resist side-slipping in situations in which some of the friction potential is required for braking or driving the powered wheels. Therefore, the possibility of tipping is reduced. Tipping could become a problem when high coefficients of friction are developed, such as on paved, dry road surfaces. It could also occur in cases in which road conditions permit higher speeds on curves. No model evaluated in this study considered tipping as a limiting factor for truck speeds.

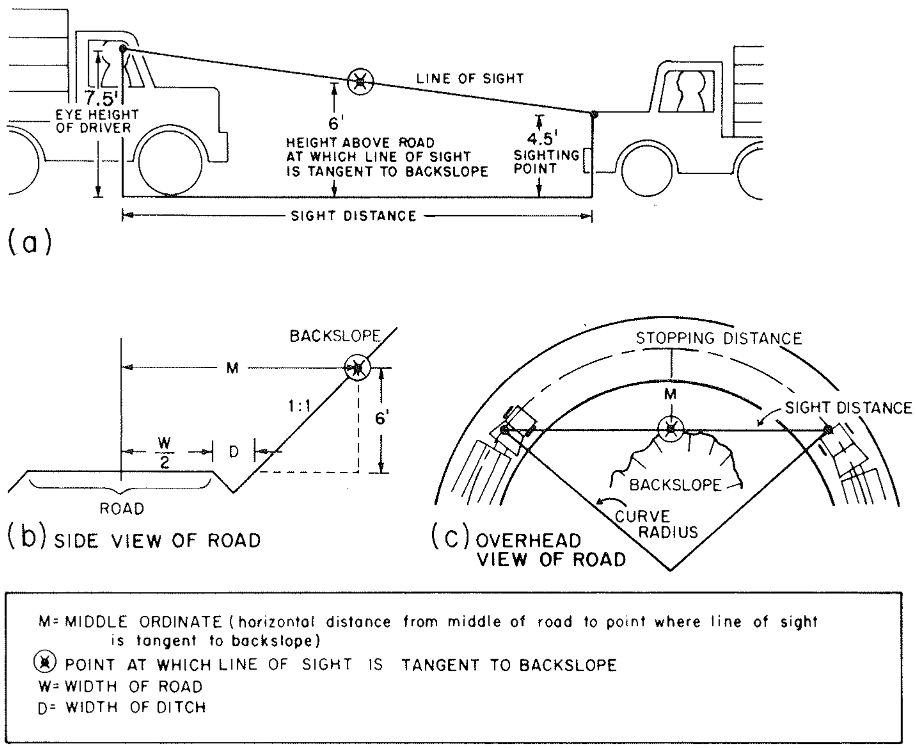

FIGURE 2 Sight distance related to road design.

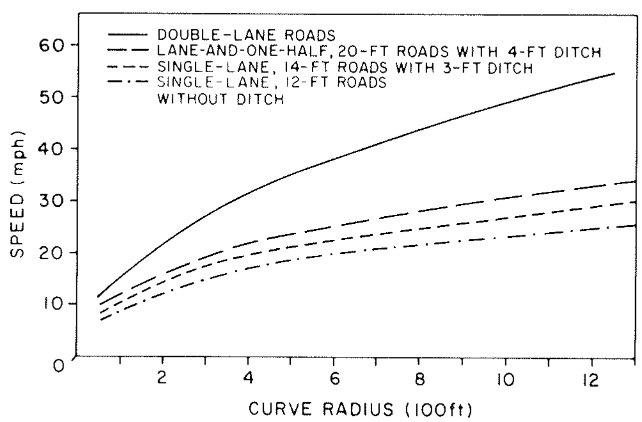

FIGURE 3 The influence of curves in logging roads of various widths on truck speed (*2*).

Interaction Between Grade and Alignment

The VOCM included the interaction of grade and curves on truck speeds, but the BNG study did not.

The approach taken by the VOCM was to compute the longitudinal and side-force friction components and compare the vector sum of these forces with the friction available between the tire and the road. The critical tires were assumed to be the powered wheels, through which the engine-braking force was transmitted. The maximum available friction was reduced by a "prudent driver" adjustment of 0.2 (*4*). For example, if the coefficient of friction was 0.4, the adjustment would reduce this value to 0.2.

STUDY AREAS AND PARTICIPANTS

For the sake of comparison with the previously described BNG study and the VOCM, truck performance was evaluated on roads in three portions of western Oregon: the 4711 road area in the Umpqua National Forest, approximately 30 mi east of Roseburg; the Wren timber sale area in the Mount Hood National Forest, approximately 20 mi east of Estacada; and the Dean Creek area on Oregon state land 5 mi east of Reedsport (Figure 4). The 4711 road area contained 20 segments that were

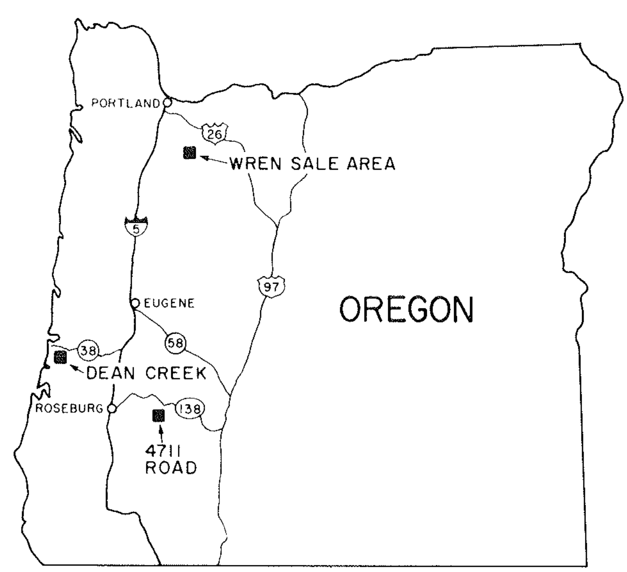

FIGURE 4 Location of study areas in which truck travel time data were collected.

located over a 7.71-mi portion of road. Only one segment was studied in the Dean Creek area. A total of 23 road segments, the lengths of which varied from 97 to 328 ft, were selected for their range of favorable grades (2 to 19 percent) and curve radii (68 to 500 ft). All roads were surfaced with 3/4-in minus gravel.

Data were collected for each road segment in the field on the grade, width, ditch depth, superelevation, and sight distance. Curve radii were obtained from plans. Information on the make, model, age, and maximum engine-braking horsepower for selected trucks participating in this study was supplied by the owners. Travel time, as well as the time of day, was recorded for each trip. Twenty-one drivers, whose truck-driving experience ranged from 10 to 30 yrs, participated. They were either employed by trucking companies that paid an hourly rate, or they owned their own trucks and were paid according to the gross weight of logs hauled.

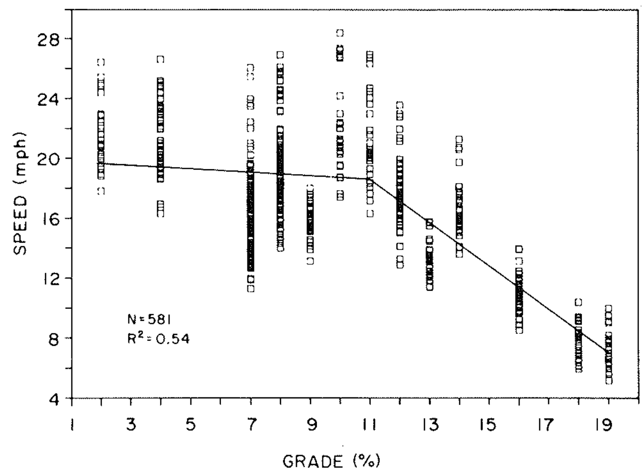

FIGURE 5 The effect of grade on the speeds of loaded trucks.

METHODS

Travel times for loaded trucks traveling downhill were recorded for each individual road segment studied in all three areas. Times were also recorded for the return runs of the empty, unloaded trucks traveling uphill. These travel times were evaluated to determine speeds as a function of curve radius and grade. In addition, overall travel time was recorded over a 7.71-mi section of the 4711 road. These times were compared with those predicted by the BNG study. Estimates of the mean and standard error were developed, and 95-percent confidence limits of these means were obtained. The relationship of the central (interior) angle to the curve radius for the 4711 road was compared with the roads in the BNG study. Multiple linear regressions were run to determine the effects of curves and grades on vehicle speeds.

RESULTS AND DISCUSSION

Effect of Grade on Downhill Speeds of Loaded Trucks

The scatter plot for grade versus speed suggested that a nonlinear transformation of the data would be necessary. No transformations using principles of mechanics could be found to fit well. A stepwise linear approach was found to give the best single fit of the data based only on grade (*6*).

The stepwise regression in Figure 5 indicates that when a favorable grade is less than 11 percent, there is little effect on speeds. The regression equation obtained was as follows:

$$\text{SPEED (mph)} = \underset{(0.058)}{19.912 - 0.123\,(\text{GRADE})} - \underset{(0.1102)}{1.329\,(\text{GRADE} - 11\%)\,(X_2)} \quad (3)$$

where

X_2 = a 0 to 1 variable (1 if grade ≧ 11, 0 otherwise) and grade is a positive number.

The break point was determined by repeated stepwise regression runs at 10, 11, and 12 percent. These break points were suggested by the scatter plot for grade versus miles per hour and were evaluated by observing R^2 and the mean square error terms for each run.

Downhill, loaded-truck speeds were slightly affected by grades less than 11 percent (Figure 5). Favorable grades steeper than 11 percent strongly influenced truck speeds. The most probable explanation is that curves influence the speeds on roads in which the grade is less than 11 percent. This is typical of many logging roads, like the 4711 road in this study, the alignment of which would be classified as poor by the BNG system (*2*). When sites were selected for this study, it was difficult to find any road sections that were not influenced to some degree by alignment.

It appears that the BNG and VOCM methods both predict speeds reasonably well for favorable grades between 11 and 16 percent. For steeper grades, the observed speeds were slower than would be predicted by both methods. In this study, because of the poor alignment of the roads, no conclusions could be reached on speed versus grade for grades below 11 percent.

Effect of Grade on Uphill Speeds of Unloaded Trucks

Travel times for the returning, unloaded piggyback trucks were also regressed on grade in a similar manner. A stepwise regression also best fit the data in this situation. The stepwise model was the same used for the loaded trucks, and the resulting regression equation in which only grade was used was as follows:

$$\text{SPEED} = 21.433 + \underset{(0.089)}{0.077\,(\text{GRADE})} - \underset{(0.172)}{1.794\,(\text{GRADE} - 11\%)\,X_2} \quad (4)$$

where

X_2 = a 0 to 1 variable, as shown.

The travel times were more variable for the unloaded piggyback trucks than for the loaded trucks; R^2 was 0.39. Grade seemed to affect the travel times of these trucks when they were traveling uphill as much as it affected the times of the loaded trucks traveling downhill. The plot of this equation and the data points are shown in Figure 6.

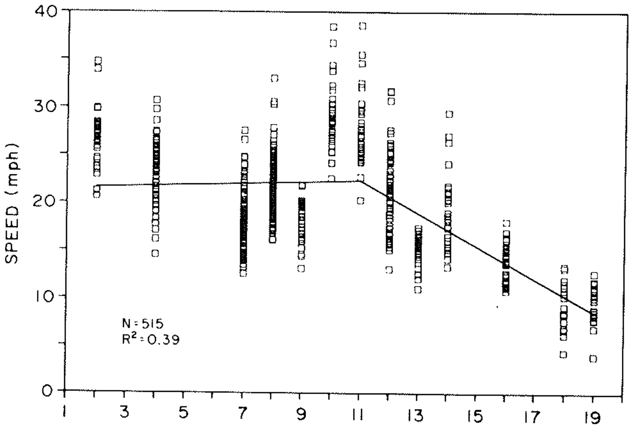

FIGURE 6 The effect of grade on the speeds of unloaded trucks.

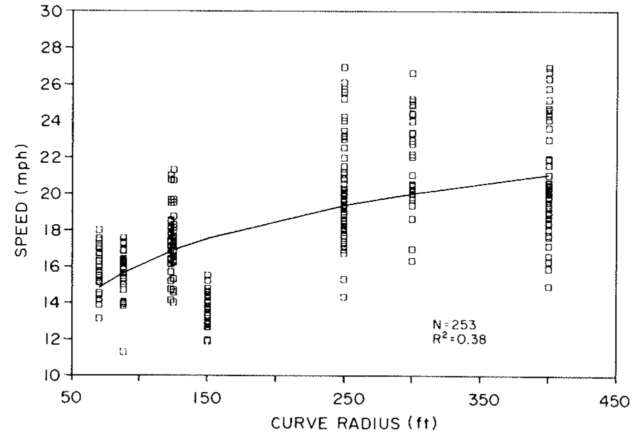

FIGURE 7 The effect of curves and grades of less than 11 percent on the speeds of loaded trucks.

Uphill travel speeds could be expected to be limited by alignment, traffic, grade, and other factors. If grade was the only limiting factor, the horsepower-to-weight ratios for these unloaded piggyback trucks would permit uphill travel at nearly 38 mph. The data offer no explanation for the similarity in speeds between the unloaded and the loaded trucks.

Effect of Curves on Speeds

Because a grade below 11 percent had little effect on speeds, only curves on portions of the road with grades less than that were regressed to estimate the relationship of a grade-free curve radius on speeds. Some interaction between grade and curve radius was present in the data, however. Curve radius decreased with an increase in grade. This relationship would normally tend to magnify the effect of curves on speeds, because as grades became steeper they would reduce speeds even more. However, little or no effect on speeds was found for favorable grades less than 11 percent.

Loaded Trucks (Downhill)

A logarithmic transformation was used to best fit the curve data for loaded trucks. The regression equation obtained was as follows:

$$\text{SPEED (mph)} = -0.373 + \underset{(0.660)}{8.239}\,[\text{LOG}_{10} \cdot (\text{CURVE RADIUS})] \quad (5)$$

The plot of this equation, with the data points, is shown in Figure 7.

Unloaded, Piggyback Trucks (Uphill)

A square-root transformation best fit the data for unloaded piggyback trucks on curves. The regression equation obtained was as follows:

$$\text{SPEED (mph)} = 11.800 + \underset{(0.055)}{0.616}\,(\text{CURVE RADIUS})^{0.5} \quad (6)$$

The variance was once again greater than for the loaded trucks, although the differences were not quite so great. The plot of this equation, with the data points for unloaded piggyback trucks on curves, is shown in Figure 8.

As mentioned earlier, the speeds of the unloaded piggyback trucks on sections of curved road were the same as those of the loaded trucks.

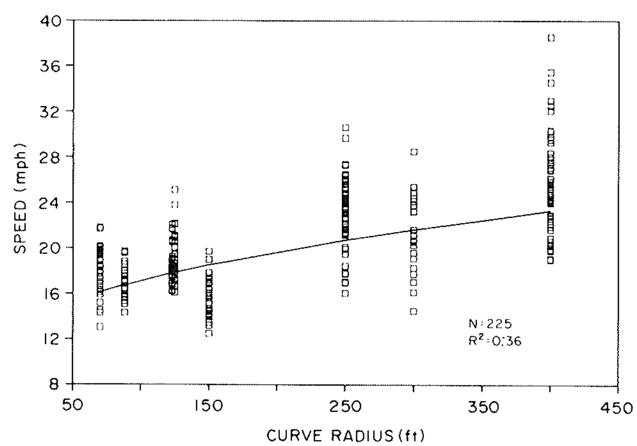

FIGURE 8 The effect of curves and grades of less than 11 percent on the speeds of unloaded trucks.

Comparison of Observed Times on Road Segments With the BNG Study and the VOCM

The plot of the stepwise regression equation for speeds of loaded trucks on grades compared with speeds predicted by BNG and the VOCM is shown in Figure 9. The curves of BNG and VOCM appear very similar and diverge at the flatter grades. Although the slope of the regression line is steeper, above 11 percent, the BNG and VOCM methods both accurately predicted speeds for favorable grades between 11 and 16 percent. Travel speeds appear to fall off more rapidly than BNG or VOCM would predict. A default speed of 55 mph was assumed as the upper limit of vehicle speed. The regression line for grades below 11 percent shows the insensitivity of speeds to grade in the range from 2 to 11 percent.

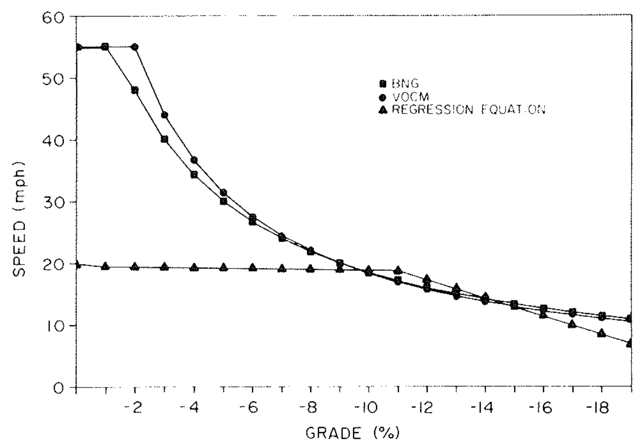

FIGURE 9 Comparison of observed speeds of loaded trucks on grades, with speeds predicted by the BNG study and the VOCM.

The plot of the stepwise regression equation for speeds of loaded trucks on curves compared with speeds predicted by BNG and the VOCM is shown in Figure 10. The curve shown for the VOCM reflects a zero grade. This relationship between curve radii and speed indicates that truck speeds were less sensitive to increasing radii than either the BNG or the VOCM predicted. The slope of the regression equation produced is somewhat flat above a 150-ft radius. Truck speeds for road 4711 do not appear to be affected by changes in curve radii as much as might be predicted by BNG or the VOCM.

The assumption that sight distances control vehicle speeds was not valid for the roads used in this study. A major contributing factor may be the extensive use of CB radios by the truck drivers. These radios, in effect, extended the sight distance in the curves by allowing the drivers to "see" ahead.

Under these circumstances, it might be expected that drivers would maintain speeds that depended on the type of road surface. However, the VOCM method, which uses this assumption, also did not agree with the observed speeds. This may be due in part to the distribution of curves on the entire road. A driver might not accelerate to a speed permitted by a 300-ft curve when a 150-ft curve is just ahead. According to the BNG

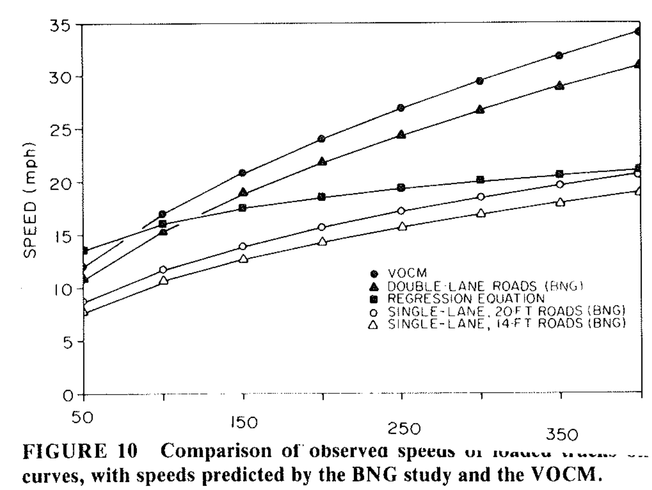

FIGURE 10 Comparison of observed speeds of loaded trucks on curves, with speeds predicted by the BNG study and the VOCM.

study, alignment class is calculated by using only curves less than four times the smallest curve radii present. The "look ahead" relationship may be the basis for the four-times rule.

Road conditions immediately ahead of or behind a segment could reduce speeds in that segment if the truck has to accelerate from a slower segment or has to decelerate before approaching a slower segment. An exception to this might be a segment on an approach to a momentum grade, where short-term speed increases could be quickly dissipated on the adverse grade ahead. Road conditions adjacent to the segment would also be expected to produce more variability in observed speeds for a given curve radius or grade. This is due to the wide range of possible road conditions immediately ahead of or behind a segment of road.

When the BNG study and the VOCM model speeds in individual curves or on grades, road conditions ahead of or behind the segment being evaluated are not included. For this reason, the comparisons made in this study are probably conservative. If adjacent road sections had affected the observed speeds, the differences between the observed speeds and the speeds predicted by the BNG study and the VOCM would be even larger.

Several drivers mentioned the importance of planning evasive action when driving on curves. Because this road was wide enough at the curves, the drivers of two meeting trucks could sometimes pass by each other, provided each kept to his own side of the road. This sometimes resulted in the use of a shallow ditch or other surface that was not actually part of the normal road width.

Comparison of Observed Times on Road 4711 With BNG-Predicted Times

In order to compare overall observed times with BNG-predicted times, the minutes required for traveling over the 7.71-mi portion of Road 4711 were recorded. According to the BNG method, either grade or alignment controls vehicle speed (*2*). Under the influence of grade, an average estimated speed was computed to be 2.71 min/mi (22.1 mph) for each loaded truck. The returning time for each unloaded piggyback truck was calculated to be 1.60 min/mi (35.5 mph). Under the influence of alignment, the speeds for the loaded and the unloaded piggyback trucks on Road 4711 were the same: 3.66 min/mi (16.39 mph). These times were delay-free.

Because unloaded trucks are expected to give the right-of-way to loaded trucks, an adjustment of 4.2 percent was added to the travel time of the unloaded piggyback truck for a total of 3.81 min/mi (15.75 mph) (*3*). The times were greater on curves than on grades; therefore, alignment was assumed to be the influencing factor on this road. The total round-trip time was obtained by adding the slower times in both directions.

The earlier mean speeds predicted by BNG are compared with the mean speed and 95-percent confidence limits for the observed data in Figure 11. It is shown in this figure that the mean speeds predicted by BNG are outside the 95-percent confidence limits of the observed data of this study, and therefore are quite different from them.

The estimate of the differences between speeds on curves is believed to be conservative because the calculations for speeds from BNG are based on a constant speed on curves and grades (*2*). These calculations do not include the acceleration or

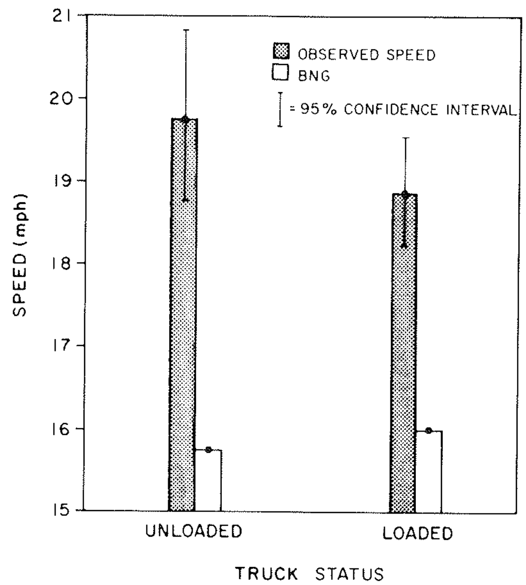

FIGURE 11 Comparison of observed speeds of loaded and unloaded trucks, with speeds predicted by the BNG study.

deceleration present in this study. If these factors were included, BNG would predict even slower times. Obviously, that would make the differences between BNG predictions and observed times even greater.

When the relationship of central angle to curve radius for Road 4711 was compared with the road in the BNG study, the two roads were found to be similar (*3*). They are probably representative of many logging roads in the West.

Overall, empty trucks were observed to travel an average of 4.4 percent faster than loaded trucks. However, the variation was great enough that no statistically significant difference between the speeds of loaded and unloaded trucks could be determined. Speeds of the empty trucks were 20.2 percent faster than those predicted by BNG, whereas the speeds of the loaded trucks were 13.1 percent faster than those predicted (*2*). Travel speeds for both empty and loaded trucks appeared to be approximately normally distributed.

The BNG method was generally found to underestimate travel speeds by 16.7 percent on Road 4711. For the 8.2-million-board-feet in this timber sale, this error is equivalent to a $1.27 per thousand board feet, or 20-percent, overestimation of hauling costs for a section of road similar to the 7.71-mi segment studied. That overestimation amounts to a total of $10,395 for the entire appraised sale volume.

SUMMARY AND CONCLUSIONS

When favorable grades were below 11 percent, they slightly influenced travel speeds. When grades were above 11 percent, they strongly affected speeds. Because of the types of road segments studied, no predictions could be made from the BNG method or the VOCM for speeds on favorable grades less than about 11 percent. The predictions of the two models and the values predicted by our regression equation are reasonably similar for favorable grades between 11 and 16 percent. Above 16 percent, however, BNG and VOCM overestimate travel speeds. Because the BNG method was primarily based on data below 12 percent, with only one point at 16 percent, it may give questionable results beyond 16 percent. The equation derived in this study predicts speeds on favorable grades above 11 percent reasonably well.

A widely accepted explanation for speeds on curves in single-lane roads is related to sight distance (*2, 5, 7, 8*). Travel speeds on curves on Road 4711 did not follow this explanation. Measured sight distance was not found to be an important variable in predicting speeds; the assumption by BNG that sight distance controls speed did not appear to be valid. It is suggested that the primary reason is the extensive use of the drivers' CB radios. These radios allowed the drivers to extend their "sight distance" to permit faster speeds in curves, with a higher degree of safety. This assumption was not tested because all truck drivers used their radios.

The relationship between curve radii and truck speed indicates that speeds were less sensitive to increasing radii than was predicted by either BNG or the VOCM. These findings could have important consequences for future road design and construction. If the presumed effects of various road geometry factors on speed are not valid, then the benefits of increased hauling speeds gained from alignment improvements may not be fully realized.

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Vehicle Tracking Simulation in Low-Volume Road Design

BRIAN W. KRAMER

Accurate determination of the horizontal surface geometry of low-volume roads in steep, difficult terrain is a critical design element. The accurate assessment of this element is directly related to the efficiency of road construction cost and vehicle operations and safety. Because traditional methods used to determine the minimum required, horizontal road surface width were inadequate, a scale model drafting device, the Drafting Vehicle Simulator, was developed. This device was developed and implemented by the USDA Forest Service, Pacific Northwest Region. The application of this design aid has yielded considerable cost savings on specific project designs when compared to traditional design procedures. The Drafting Vehicle Simulator accurately simulates tire-mounted, Ackerman-steered, and nonarticulated vehicles traveling at low speeds. Single-unit, multi-unit, and special vehicles can be modeled. The simulator can be applied to the design analysis of horizontal road geometry, such as tracking through simple and compound curves, intersections, and approaches to road structures, and backing up and turning around. The Drafting Vehicle Simulator is a low-cost, easy-to-operate, and portable low-volume road design aid that yields accurate results. It has applications for the design of civilian and military low-volume roads and analysis of existing roads for vehicle passage.

Thousands of miles of low-volume roads are constructed worldwide each year. These transportation facilities are required to accommodate many different types of vehicles. Efficient road design is necessary to minimize construction cost and provide safe, adequate transportation facilities. Road construction costs are minimized when a minimum road design geometry is applied to accommodate design vehicles safely and efficiently. In order to meet the challenges of increasing forest road costs, the USDA Forest Service implemented a national program in 1981 to reduce the agency's road construction costs. One major objective of this program was to develop minimum geometric design standards for low-volume forest roads. To help meet this objective, a scale model, the Drafting Vehicle Simulator (DVS), was developed and implemented for use in national forests in the Pacific Northwest region. This device is a road design aid that is used to analyze the tracking characteristics of various design vehicles that operate on forest roads to determine the minimum required, design road surface width geometry.

The following three major objectives were achieved in the development and implementation of the DVS:

- Accurate vehicle tracking simulation of a wide range of large vehicles that use forest roads in the Pacific Northwest,
- Simple, efficient operating procedures with cost-effective results, and
- Low-cost equipment procurement and maintenance.

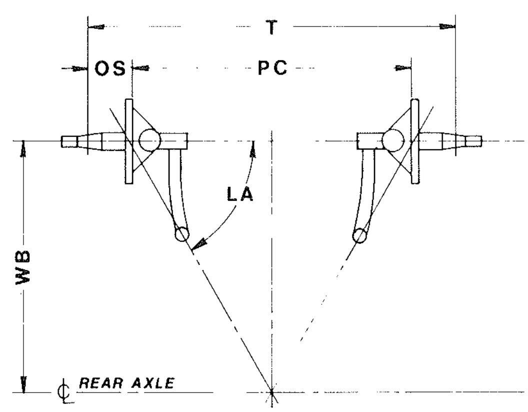
The DVS has several advantages to existing scale and mathematical models used to estimate vehicle tracking characteristics. These advantages are summarized in Table 1.

The DVS is fabricated from brass and stainless steel key stock. All parts are machined to tolerances of ± 0.001 in. The steering mechanism and all wheels are constructed with precision ball bearing sets. The tires are rubber O-rings. It is designed to operate at a minimum scale at which 1 in equals 10 ft.

The DVS accurately simulates and plots vehicle tire tracking of a nonarticulated, Ackerman-steered (Figure 1), and tire-mounted vehicle traveling at low speeds. Excellent tracking correlation was obtained with the tracking data developed by the California Department of Transportation using a Tractrix Integrator, which is a scale model of a tractor-trailer unit.

The DVS and accessory components are stored in a water-tight, crush-proof carrying case to facilitate field transport and storage (Figure 2). The tracking characteristics of the following vehicles are easily simulated:

- Any single-unit vehicle. One example is a large log yarder with a forward down-rigged tower (Figure 3).
- Highway tractor-trailer (Figure 4).
- Tractor-jeep-trailer (Figure 5).
- Log truck (Figure 6).



T Track of tires at ground

PC Distance between knuckle pivot centers (True)

O.S. Offset, pivot center to track of tire on ground

W.B. Wheelbase

La Cross steering lever angle from axle centerline (True)

FIGURE 1 Basic Ackerman steering arrangement.

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TABLE 1 COMPARISON OF FOUR VEHICLE TRACKING SIMULATION MODELS

	Computer Model	Scale Drafting Models		
Simulated Parameters	USFS Tractrix Equations	U.S. Army Vee-Tran	USFS DVS	California DOT Tractrix Integrator
Maximum steering cramp angle?	No	No	Yes	No
Backup and turning maneuver?	No	No	Yes	Yes
Track through compound curves?	No	No	Yes	Yes
Simulate various standard highway units?	No	Yes	Yes	No
Simulate log yarding equipment?	No	No	Yes	Yes
Simulate a log truck?	No	No	Yes	No

FIGURE 2 Drafting vehicle simulator and accessories in storage case.

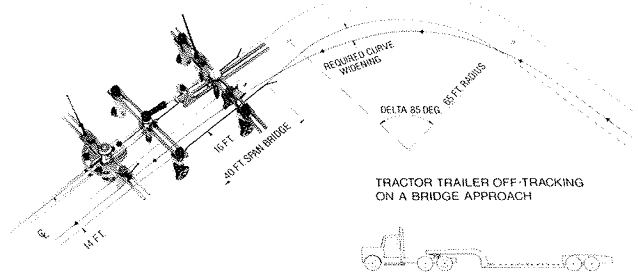


FIGURE 4 Tracking plot of a highway tractor-trailer traversing a simple curve and bridge in one direction.

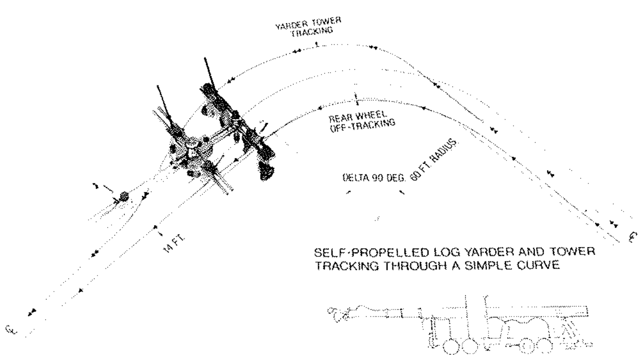


FIGURE 3 Tracking plot of a large Pacific Northwest log yarder and tower traversing a simple curve in one direction.

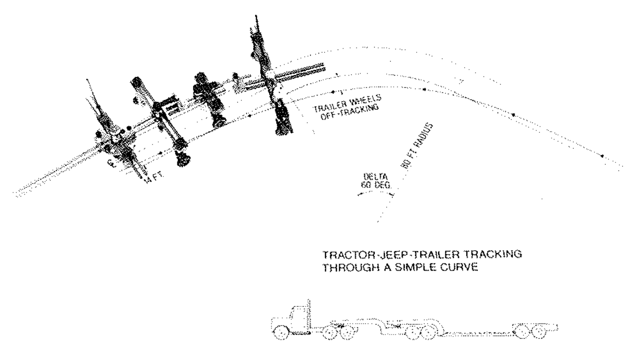


FIGURE 5 Tractor-jeep-trailer traversing a simple curve in one direction.

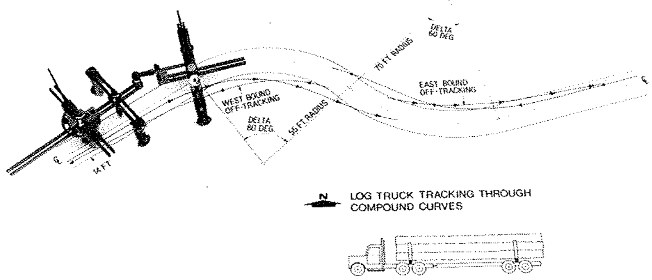
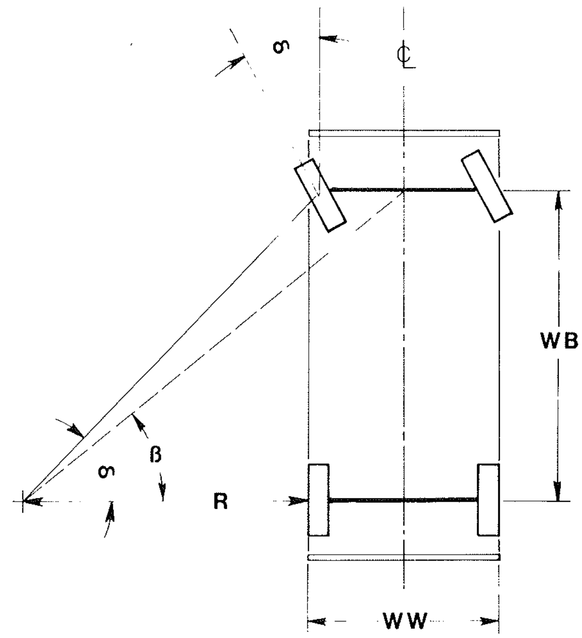


FIGURE 6 Log truck traversing a reverse and compound curve in two directions.

The DVS can be adjusted to a particular vehicle scale with hand-adjusted thumb screws and an Allen screwdriver. The steering unit's cramp angle, button-head stops can be adjusted in 5° increments from 5 to 45° left and right (Figure 7). The DVS was designed with a tricycle steering arrangement. In order to simulate Ackerman steering, the maximum Ackerman steering cramp angle must be recomputed and the DVS cramp angle stops must be adjusted accordingly (Figure 8). A 6-in plastic scale with increments of 10 to 1 in is provided for wheel base and vehicle overhang adjustments. Ball-point pens are used to plot wheel and vehicle overhang tracking. Out-to-out vehicle wheel widths are most easily scaled by plotting these dimensions on 10- to 1-in engineering paper and making appropriate adjustments (Figure 9).

The DVS is easily operated by moving the model through a section of road and plotting the surface width on drafting paper to a scale at which 1 in equals 10 ft (2.54 cm equals 3.05 m). The unit is propelled and steered by the steering knob on the steering unit. The road designer simulates actual steering through curves as if he was driving the simulated vehicle.



WW Out to out wheel width

WB wheel base

β Vehicle maximum steering cramp angle

σ D.V.S. simulated maximum steering cramp angle

$$\tan^{-1} \frac{WB}{R \quad \frac{WW}{2}}$$

FIGURE 8 DVS steering cramp angle computations.

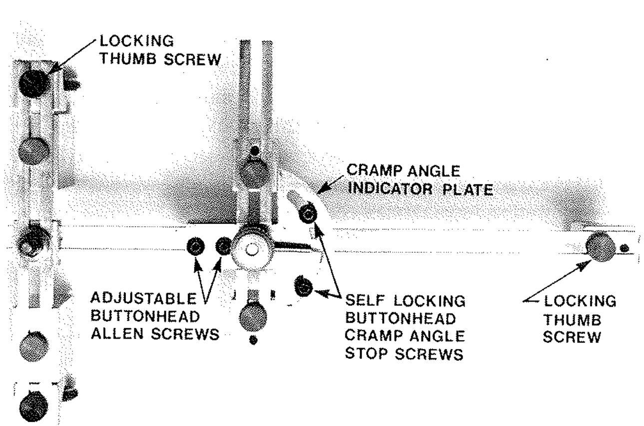


FIGURE 7 DVS steering unit scale adjustments.

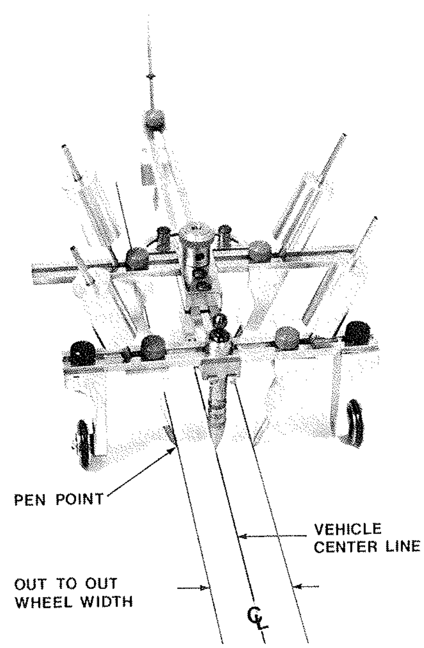


FIGURE 9 DVS wheel width adjustment.

The following are examples of DVS road design project applications:

- Vehicle tracking characteristics through simple and compound curves to determine minimum required curve widening,
- Restricted operating and parking spaces for tractor-trailer units,
- Critical bridge approach designs,
- Minimum parking facility design required for large tour buses,
- Empty log truck turn-around designs,
- Waterfront log dump design,
- Analysis of log sweep of tree length logs on a log truck on curves and in intersections,
- Large log yarder wheel tracking and tower sweep to determine minimum road width and cut bank design, and
- Required curve widening for design of roadway retaining walls.

The procurement cost of the DVS was a prime consideration in its development and design. The cost of the entire unit, including the carrying case, was $495.00 in 1985.

The DVS has proved to be a cost-effective design aid in the analysis of vehicle tracking in low-volume roads. Its application covers a wide range of practical vehicle tracking design situations that other scale and computer models do not effectively address. The operating procedures of the DVS are easily learned and rapidly applied. A road design engineer can be taught to use the DVS in 1 hour. The application of this tool in road design yields considerable savings in road construction costs when compared to the traditional design analysis of horizontal road geometry.

Development of New Design and Construction Guidelines for Low-Volume Road Bridges

HOTA V. S. GANGARAO AND MICHAEL J. HEGARTY

Low-volume road bridges in the United States are currently designed with the aid of AASHTO Standard Specifications for Highway Bridges. These specifications were primarily developed for high-volume urban and interstate highways. The design and construction of low-volume road bridges is therefore expensive. It is obvious from the current design and funding pattern in the United States that available funds are not sufficient to rehabilitate or replace all the deficient low-volume road bridges. A systematic investigation is therefore being performed to study the cost-effective use of various super- and substructural systems and miscellaneous bridge components to better use available funds. The unique characteristics of a low-volume road bridge are defined in this paper as a function of speed limits, average daily traffic, gross vehicle weight, and bridge width. The standard highway bridge specifications of the United States and other countries are reviewed and modifications to certain specifications and elimination of others are proposed. Feasible low-volume bridge components can be selected by eliminating inappropriate alternates and comparing the advantages and disadvantages of the remaining super- and substructural bridge systems. Concrete, steel, and timber structural components are reviewed, and design and cost scenarios that were developed in this study are highlighted. In order to recommend the effective use of limited available funds for bridge replacement programs, a value engineering approach was adopted to research the cost-effectiveness of various bridge components; types of materials used in the construction of low-volume road bridges; and current specifications for design, construction, maintenance, and rehabilitation.

Over the past several decades, about a trillion dollars have been invested in the highway system of the United States. However, a massive infusion of additional funds is required to maintain, rehabilitate, and replace the rapidly deteriorating highway system. For example, it is estimated that $48.9 billion in 1982 dollars will be needed to repair or replace 253,196 of about 600,000 bridges that were classified as deficient at the end of

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1982 (*1*). It is obvious that available funds ($7.05 billion for 4 years through 1986, as authorized by the U.S. Congress) are insufficient to rehabilitate or replace all deficient bridges.

The major objective of this study is to evaluate the specifications and design criteria for bridges and determine if certain aspects of the existing criteria can be eliminated or modified to make low-volume road bridges more cost-effective. Low-volume roads are defined as those roads that have an average daily traffic (ADT) of less than 200 vehicles, and a posted speed less than 35 mph. Another objective is to systematically investigate the economics of various low-volume super- and substructural systems and miscellaneous bridge components, such as curbs, railings, and expansion joints. It is hoped that the potential savings derived through the proper selection of innovative bridge systems can lead to better use of available funds. This is particularly important in regard to badly deteriorated old bridges the life of which cannot be extended by maintenance alone.

The initial phase of the comprehensive work being conducted at West Virginia University is reported. In-depth investigations of the proposed modifications to develop new specifications for low-volume bridges will be reported as a sequel to this study. Because the current categories of bridges are wide and all-encompassing, only such items as material selection, design criteria, construction type, and maintenance and rehabilitation costs were carefully identified and evaluated before specifications were formulated to optimize the design of a low-volume bridge.

STATE OF THE ART

A comprehensive review of various technical articles on low-volume bridges and the associated topics was performed. It was found that the unique characteristics and problems of low-volume bridges were presented in specific, narrow subcategories (i.e., precast and prestressed, concrete low-volume bridges) and little comprehensive work was presented on the general topic of low-volume bridges.

Consequently, it is not surprising that the AASHTO design specifications do not differentiate between low-volume rural bridges and high-volume urban bridges (*2*). It is therefore highly unlikely that efficient and economical low-volume bridges can be designed using specifications that were compiled primarily for highway bridges. It should be noted that the only section of the AASHTO code that considers the ADT of a bridge is the section on allowable fatigue stresses. The Ontario Highway Bridge Design Code also does not distinguish between high- and low-volume bridges (*3*).

Much of the pertinent material found under other subcategories includes material type, relative economic comparisons, systems construction approach, jointless bridges, and use of guardrails and curbing. The available literature under each subcategory was reviewed in regard to specifics concerning low-volume bridges.

Material Type

Many useful low-volume bridge applications were found in articles that discussed prestressed concrete, timber (glulam), or steel alternatives. Precast, prestressed concrete is applicable as a construction material in low-volume bridges because it can be prefabricated and is economical in many regions of the country. Tokerud considered a wide range of issues that affect precast and prestressed, low-volume concrete bridges, including planning, design considerations, abutments, bridge decks, and geometrics (*4*). A list of design and construction recommendations is also provided by Tokerud.

Precast concrete bridge decks are also a viable alternative to conventional bridge decks, especially for low-volume bridges (*5, 6*). Berger noted that modular precast decks have been successfully used since 1967 with excellent results (*7*). These deck modules are mass-produced because the same form work can be used repeatedly; quality control is improved because on-site pouring of concrete is eliminated. Other advantages include greater structural efficiency and a possible reduction in dead load. Construction cost data for four design examples were estimated and compared with conventional cast-in-place construction. Sprinkle stated that the use of these decks reduces the on-site construction time, which provides savings in labor costs (*8*).

Timber construction, especially glued-laminated (glulam) timber construction, lends itself well to low-volume bridge construction (*9, 10*). The fact that these components can be prefabricated means that savings similar to those derived from the use of precast concrete will result. Verna presented three case histories that outlined the rehabilitation of various components using glulam members (*11*). Verna stated that timber exhibits acceptable resistance to deicing agents and normal water exposure, and must be considered in the selection of bridge materials. Other authors expressed concern about such factors as installation problems with the alignment of deck panel dowels and cracking of any asphaltic overlay (*12, 13*). However, these problems are being resolved because of a greater understanding of their behavior (*14-16*).

Open-grid steel decks are not used on many large urban highways because of certain undesirable characteristics, including twisting and fracture of diagonals, weld failure, and corrosion problems. GangaRao noted that the underdesign of grid deck components because of a lack of understanding was the major cause of these failures (*17*). In addition, open-grid steel decks have been noted as having poor skid resistance, which makes them unacceptable for high-speed traffic. However, because of the low speed limits on low-volume bridges, skid resistance is not a major design or safety consideration. The most obvious advantage to the use of grid decks is that they are extremely light (15 psf).

Economic Considerations

A few studies attempted to compare the economics of one bridge system to another (*18*). Hill and Shirole presented a statistical breakdown of 3,700 bridge replacements for spans of less than 100 ft in the state of Minnesota (*19*). The 10-yr (1973 to 1983) test group was broken down into bridge type, number of each bridge type, and unit construction costs.

In a more recent report by Sprinkle, prefabricated bridge elements and systems were analyzed and current practices and problems were listed (*20*). In addition, a set of tables was presented and the costs of various alternatives were quoted according to several sources.

A complete methodology to determine whether a deteriorated bridge should be rehabilitated or replaced on a minimum life-cycle cost basis was developed by Weyers, Cady, and McClure

(*16*). Cash-flow diagrams were used to determine the equivalent uniform annual costs of alternatives. The effect of interest rates and inflation was then considered to obtain the most economical solution. Examples of the mathematical model and a microcomputer program were also presented.

Tokerud studied the potential economics of low-volume bridges (*4*). He discussed the most economical bridge alternatives considered in the northwestern United States under the three structural material groups of concrete, timber, and steel. Planning, design, contracting, and geometric considerations were also discussed. This study is based on a survey of state, county, and municipal agencies in that region.

It should be noted that economic information in regard to low-volume bridges in published technical literature is limited. This is especially true in regard to future costs, such as yearly maintenance and rehabilitation costs. Such information is necessary to establish the life-cycle costs of alternatives, which is the purpose of the value engineering approach. A sequel to this study is therefore being prepared at West Virginia University, with special emphasis on a value engineering approach to select a specific bridge system.

The Systems Approach

The systems approach to the construction of low-volume bridges is desirable for several reasons (*10*). More efficient use of materials through mass production coupled with the avoidance of costly and time-consuming conventional procedures are just two of the advantages of the systems approach.

GangaRao presented 10 different substructure systems and analyzed them in regard to economy, ease of erection, maintenance, and longevity (*21*). Hanson also presented prefabricated substructural units (*22*). Sprinkle discussed systems construction techniques for short-span concrete bridges and listed several uses of prefabricated components (*8*). GangaRao and Taly developed several innovative prefabricated, superstructural systems for spans of up to 100 ft (*23*). A numerical rating scheme was used to evaluate the alternatives in relation to one another.

Use of Jointless Bridges

Expansion devices and bearings are not used in jointless bridges because thermal expansion movements are transferred directly from the superstructure to integral or hinged abutments. Additional details are provided later. The costs of jointless bridges therefore do not include the initial cost of expansion joints and bearings and associated maintenance costs.

The jointless bridge also acts as a rigid frame because the superstructure is tied securely to the substructure, which substantially reduces the moments incurred by the superstructure and substructure (*24*). The state of Tennessee has constructed a 927-ft concrete bridge and a 416-ft steel bridge (*25*). Span length therefore is not a major consideration in the design and development of a jointless bridge.

Use of Guardrails

Bronstad and Michie investigated the applicability of guardrails on low-volume bridges (*26*). They proposed that guardrails are not warranted for bridges with an ADT of less than 50. Many government agencies, such as the U.S. Forest Service, use railings that do not meet AASHTO specifications. Preliminary discussions with state highway agency personnel revealed that guardrails do not appear to serve their purpose on low-volume bridges. The necessity of guardrails on low-volume bridges is evaluated in a later section of this paper.

ISSUES THAT AFFECT LOW-VOLUME BRIDGES

The use of various alternatives to maximize the economy of a low-volume bridge is considered. The following four topics are addressed: a review of existing codes and geometrics, minimization of components, material selection, and use of guardrails and curbs.

Review of Existing Codes

Low-volume bridges in the United States are currently designed according to the same criteria as urban and interstate highway bridges. It is obvious that the specifications for a bridge designed to safely carry high-volume HS-20 loads are overly conservative when applied to a low-volume bridge. Existing bridge design and specification codes are therefore reviewed in regard to fatigue, lane loading, and deflection to identify which standards are overly conservative or irrelevant.

Fatigue

Fatigue damage is a major consideration in the design of a bridge. The standard specifications for highway bridges adopted by AASHTO define allowable levels of stress that correspond to the number of fatigue cycles the bridge will experience in its lifetime (*2*). The lowest level of design stress corresponds to a maximum of 100,000 fatigue cycles.

The following three alternatives are available to minimize the significance of the fatigue factor in regard to low-volume bridges:

- Assume the current allowable fatigue stresses listed by AASHTO for 100,000 cycles;
- Expand the current AASHTO allowable fatigue stress criteria to include another category of cycles at a lower limit (i.e., less than 50,000 cycles); or
- Neglect fatigue effects in design.

If the second or third alternative is chosen, savings would be realized by using higher levels of stress, which would result in smaller size sections for stringers.

Vehicle Impact

No changes in vehicle impact for low-volume bridges are proposed. Many studies have concluded that the prediction of the dynamic loads as a result of vehicle impact are affected by a number of variables (*27*). The current practice of making the impact factor a function of span length with a maximum value of 30 percent is recommended to be retained for low-volume roads. It should be noted that Ontario Highway Bridge Design Code uses a maximum value of 40 percent, with various reductions of that percentage specified depending on the component type (*3*). An average increase of static live load ranges of 30 percent is therefore recommended for low-volume bridges.

Lane Loads

Lane loads were developed to provide a simpler method of calculating moments and shears than methods based on wheel loads (*2*). A truck train was modeled to attain the worst design criteria. This loading combination is irrelevant because the probability of a truck train being used on a low-volume bridge is essentially zero. It is therefore recommended that an analysis of lane loadings should be omitted in the case of low-volume bridges.

Deflections

Current AASHTO practice limits deflections by using specifications that are a function of span length, whereas a second type of specification involves the use of beam depth ratios in regard to span length for steel bridges. In both cases, AASHTO notes that these limits can be exceeded at the discretion of the designer. It is noted that these ratios are primarily serviceability considerations that account for user comfort and may not affect the structural integrity of a bridge.

Higher allowable levels of deflection can be used in a low-volume bridge because of its unique characteristics. The maximum acceptable deflection currently specified by AASHTO is L/800. It would be feasible to relax the deflection criteria to the levels prescribed for building floors (L/360) because only one vehicle would be on the span at a time. The proposed L/360 requirement would replace the other associated AASHTO criteria that limit deflection by controlling beam spacing, as is the case with concrete structures.

If deflection requirements were relaxed, savings would result in several areas. Deflection considerations are likely to control the design of girders in longer spans. If the requirements are relaxed, smaller or shallower members may be satisfactory. Wide flange sections might be able to satisfy the new design criteria in cases in which cover plates or built-up sections are required.

Geometrics

Items such as bridge width, posted speed, roadway approach curvature, and clearance heights must all be reviewed from the standpoint of their cost-effectiveness in terms of the present worth of a structure. The greatest cost reductions could be realized by reducing the bridge width. The possibility of two vehicles crossing a low-volume span simultaneously is remote. Therefore, serious consideration must be given to designing a one-lane bridge with a 12- to 15-ft clearance width.

Some low-volume bridges are definitely not suitable for one-lane configurations. Restricting variables include the following:

- The use of a bridge by oversized vehicles such as farm equipment;
- High roadway speeds or dangerous roadway alignments; and
- The prospect of future development.

It would certainly be justifiable to place a one-lane bridge on a one-lane roadway, but it would be questionable to place a one-lane bridge on a lightly traveled two-lane road.

One approach would be to include additional safety provisions to compensate for the one-lane span. These provisions could include additional warning signs cautioning the motorist of the upcoming roadway change, speed bumps, and possibly the installation of guardrails. However, as will be explained later, guardrails are not economically justified on most low-volume bridges (*26*).

If the suggested safety provisions are enacted, the additional cost will be more than offset by the reduction in costs of the substructure and superstructure. A savings of almost 50 percent in materials costs could be realized if a one-lane span is chosen over a two-lane span. It is also reasonable to assume that construction costs could be decreased 30 to 40 percent by building a one-lane bridge instead of a two-lane bridge.

Clearance heights should also be reviewed. Both overhead and underpass clearances must be reviewed and revised to determine if savings in construction and maintenance costs can be realized.

Overhead clearance should generally not be a problem because it can safely be assumed that through truss or suspension bridge types would not be considered for low-volume bridges.

Bridge underpass clearance above a waterway must be determined by considering the hydraulics of a particular site. Conventional design practices are generally applicable, although low-water stream crossings that are designed to be over-topped by floods could be an economical alternative (*28*).

Minimizing Components

The careful selection of components and materials can help reduce the construction and maintenance costs of low-volume bridges. The use of monolithic, or tied-down, abutments is one major method of reducing costs. In this system, the superstructure is firmly secured to the abutments, which creates a rigid frame. A rigid frame has the following advantages:

- Lower design moments of up to 20 to 30 percent;
- Damping of dynamic (impact) forces that result from transmission through the frame and into the soil; and
- Reduction of moments at abutment base because the superstructure resists lateral earth pressures.

In addition to the use of monolithic abutments, the concept of a jointless bridge would be applicable to low-volume bridge construction. The construction of a jointless bridge could reduce several costs. The cost of expansion joints and their associated maintenance costs would be eliminated, and smaller bridge members could be used because of the rigid frame action.

Continually reinforced concrete highway pavements have been constructed for years (*29*). This concept has also been successfully applied to bridges in which the joint at which the bridge and roadway meet and all intermediate deck joints are eliminated. Thermally induced lateral loads and the vertical load are transferred directly to the integral abutment by use of monolithic construction.

Initial savings are derived by removing the joints from the bridge. However, more significant savings can be realized by the reduced maintenance cost associated with jointless superstructures. Expansion joints require periodic cleaning and inspection, and often do not function as they were intended to.

Material Selection

Careful selection and use of materials can reduce the construction and maintenance costs of a low-volume bridge. The unique characteristics of low-volume bridges must be considered when a selection is made.

As will be explained later, it is desirable to select a material with low maintenance costs even if the initial costs are higher. It is therefore beneficial to minimize, if not eliminate, the number of connectors on a bridge from both maintenance and inspection standpoints. The feasibility of a bridge with no connectors must be determined, but such new techniques as the use of epoxy glue as an adhesive must first be thoroughly investigated before they can be recommended for general use.

The use of lesser-grade materials (i.e., F_c = 3,000 psi concrete) generally does not result in appreciable net savings (*4*). However, because low-volume bridges are generally in remote regions, the lower costs of local materials could justify their use in terms of substantial savings.

Use of Guardrails and Curbs

The costs of installing bridge railings range from $10 to $80 per linear foot depending on the type of railing installed (*26*). The savings to be realized are easily in thousands of dollars if bridge railing costs could be minimized or even eliminated by considering the unique requirements of low-volume bridges. The following is a review of the issues relevant to this problem.

Only one level of bridge railing service is currently recognized by AASHTO (*2*). Although this level of service adequately provides a safe bridge railing design for large urban highway bridges, it is questionable whether or not this same level of service could produce cost-effective bridge railings for roads with an ADT of less than 10. A report published by the Transportation Research Board investigated this problem (*26*).

Comprehensive bridge accident data were analyzed in the report and the results were integrated in a cost-benefit model. It was found that railing was not economically justified for bridges with an ADT of less than 50. This conclusion was made by determining the probability of an accident occurring on a certain type of bridge and the material and human costs of that accident. This result was then correlated with the value of the estimated benefit of retaining the vehicle on the bridge deck. However, in the case of low-volume bridges, the impact probability is almost zero; therefore, bridge railing does not provide more benefits than costs on low-volume bridges.

Two low-cost railing alternatives are presented in the report, should a designer believe that specific site conditions warrant bridge railings. One is a steel post and the other is a timber post at 8-ft, 4-in centers with their beams. These alternatives were designed using a vehicle weight of 4,500 lbs, an impact speed of 60 mph, and an impact angle of 15°.

Although these criteria are less demanding than current AASHTO barrier criteria, it is proposed in this report that it is more realistic to use these criteria in the design of railings for low-volume bridges. The average cost of these alternatives is only $10 per linear foot.

SELECTION OF FEASIBLE ALTERNATIVES

The cost of a new bridge on a low-volume road is broken down on the basis of the deck, superstructure, and substructure bridge components. The most cost-effective structure can be built by ranking and maximizing the use of each component, and determining their initial and future costs.

Based on a questionnaire, low-volume bridges were defined as those that have an ADT of less than 200. These bridges are often rural bridges that are located far from maintenance crews and materials. It is therefore logical to spend more money on the initial cost if it is spent on an alternative that requires minimum maintenance, both in terms of tasks to be performed and number of visits by maintenance personnel.

Bridge Decks

Recent trends in new bridge construction have shown a definite movement away from labor-intensive and time-consuming construction (*19*). Orthotropic decks that require a great amount of skilled labor therefore can be eliminated. Concrete-filled, steel-grid decks can also be eliminated because of the costs, labor intensity, and other maintenance problems.

Three types of prefabricated bridge deck systems are suggested: precast, prestressed concrete deck panels; open steel-grid deck panels; and glulam deck panels.

These prefabricated decking systems have advantages and disadvantages. However, the key factor in determining the most economical bridge deck alternative is the location of the site to the deck system producers. Although glulam is frequently used in the Northwest, concrete is by far the most common material for decks in the East (*4*).

This same principle governs the use of cast-in-place concrete on low-volume roads. If fresh concrete can be economically hauled to a certain site, it probably is the best choice. However, it is safe to say that most low-volume bridges also are many miles from a concrete plant. Cast-in-place construction is also labor-intensive. Cast-in-place bridge deck construction is therefore probably not one of the most economical methods.

Bridge Superstructures

As was the case with bridge decks, certain types of bridge superstructures are more suitable than others for low-volume bridge construction. Among the groups that should not be considered are steel build-up sections, trusses, and cable-stayed and suspension bridges. These types of construction are highly labor-intensive and would not be economical alternatives.

Prefabrication is suggested once again because of the high probability that these bridges are located in remote areas. Therefore, cast-in-place construction is generally not economically justifiable because of the high cost of transporting materials and labor to the job site. This may not be true in some situations, and cast-in-place concrete construction might be a viable alternative.

Another superstructure type that is not suggested is a precast, reinforced concrete system. This method of construction is generally used on short span lengths of less than 30 ft. Highway agencies may be able to make their own precast members by using idle construction workers in winter months. Because prestressed, precast members are structurally more efficient than their reinforced, precast counterparts, they only are considered in part of this study.

The three most promising types of bridge superstructures are precast, prestressed concrete stringers; steel stringers; and glulam stringers. In addition, a wide range of deck stringer systems has been analyzed from the standpoint of their applicability for low-volume bridges, including the following:

- Prestressed or nonprestressed plank timber or glulam decks with steel or glulam stringers (Figures 1 and 2);
- Voided slab (Figure 3);
- Cast-in-place concrete or glulam decks with steel or glulam stringers (Figure 4);
- Prestressed or steel-grid deck panels with steel stringers (Figures 5 and 6);
- Cast-in-place deck with precast, prestressed I-beams (Figure 7);
- Precast decked bulb T-beams (Figure 8); and
- Box beams (Figure 9).

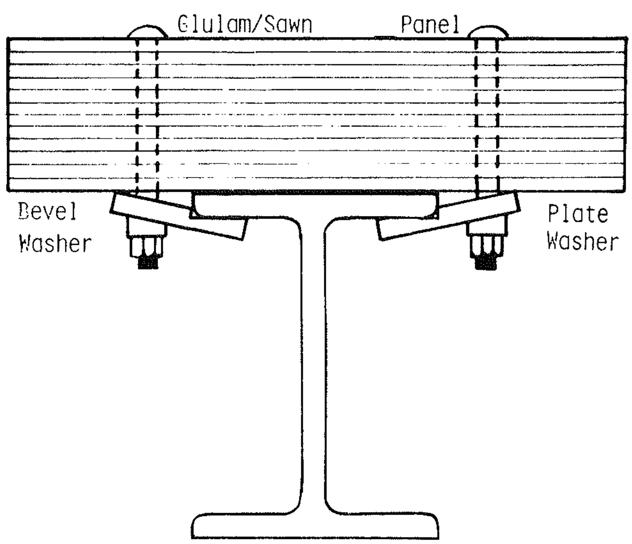

FIGURE 1 Glulam/sawn panel with steel stringer.

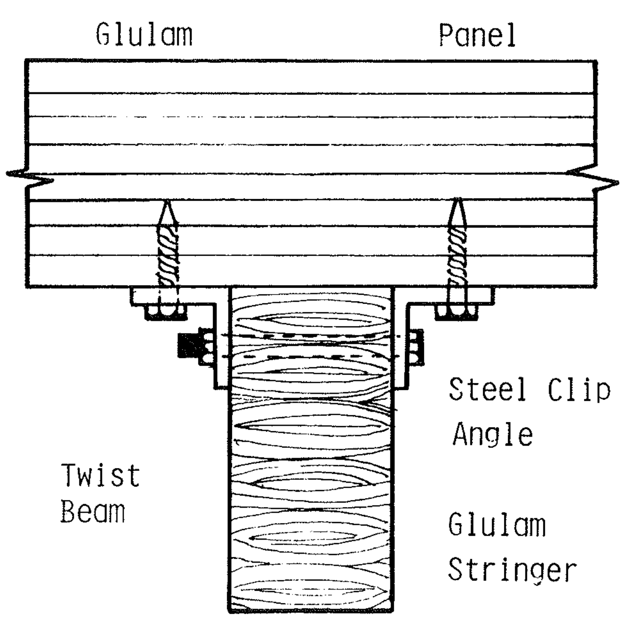

FIGURE 2 Glulam stringer and panel system.

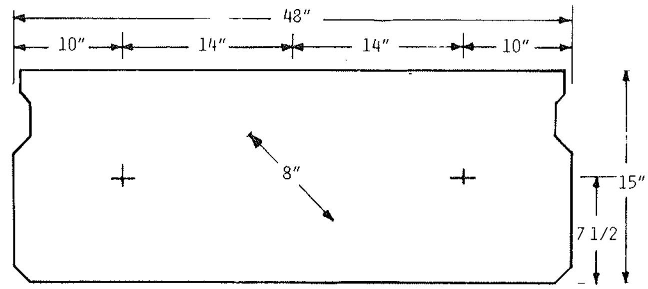

FIGURE 3 Voided slabs for a 30-ft span.

The first two items in the list are commonly considered for span ranges of 30 ft and less, whereas the rest of the items are considered for spans ranging from 60 to 100 ft. A few cross-sectional details of these bridge systems are given in Figures 1 through 10.

Other ways to minimize cost of the superstructure include avoiding the use of diaphragms and other projections, limiting skews to less than 30°, using welded wire fabric and elastomeric bearing pads when possible, and repeating a great number of identical spans when possible.

Bridge Substructures

Finally, potential savings can be derived from the choice of abutment type. The use of either a full or stub abutment can significantly affect the final cost of a bridge. The use of full abutments involves the construction of vertical abutment walls that are backfilled to create a level subbase. The use of this

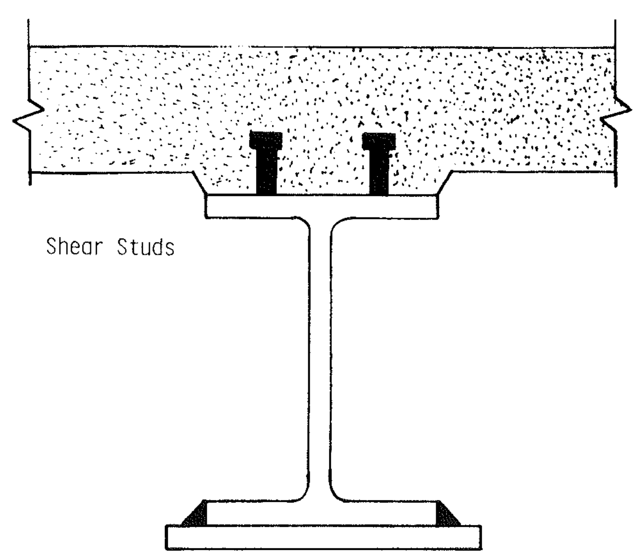

FIGURE 4 Composite WF-section with cover plate.

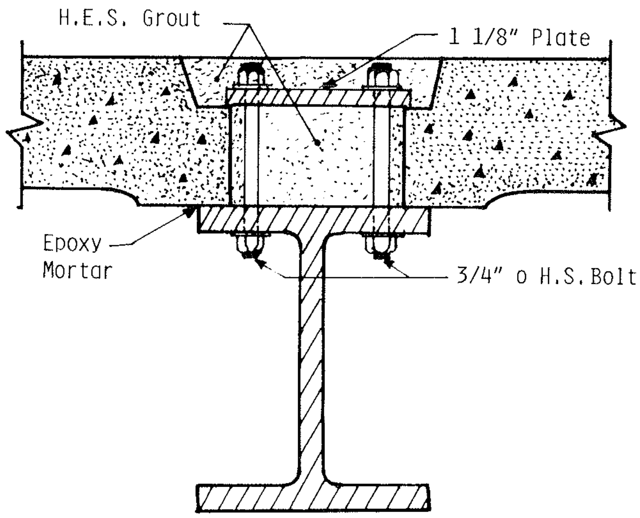

FIGURE 5 Precast deck panel steel stringer system (grouted connection).

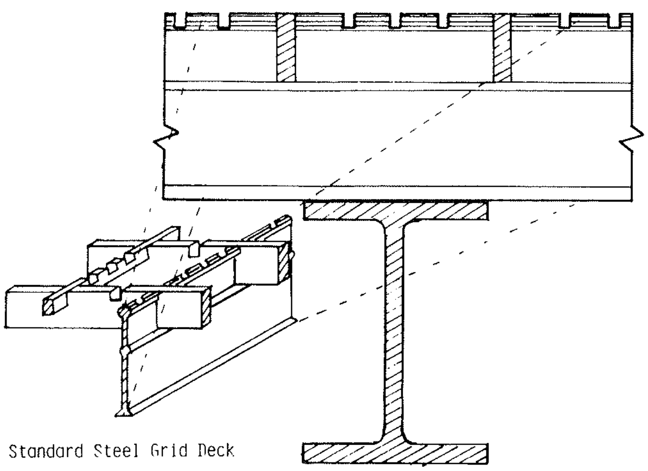

FIGURE 6 Open steel-grid stringer bridge system.

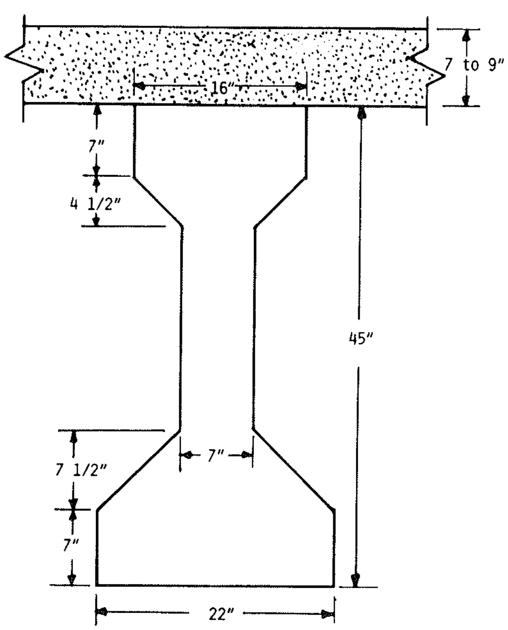

FIGURE 7 Type III girder section.

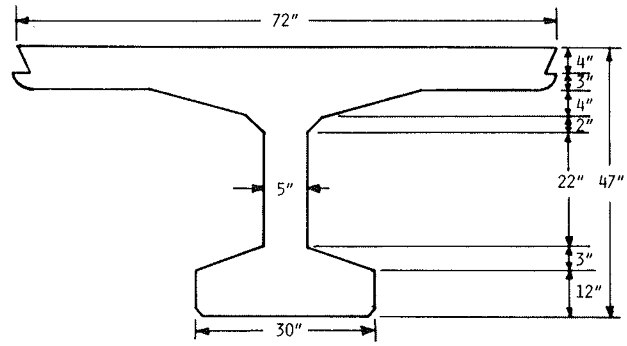

FIGURE 8 Decked bulb T-beam for 100-ft span.

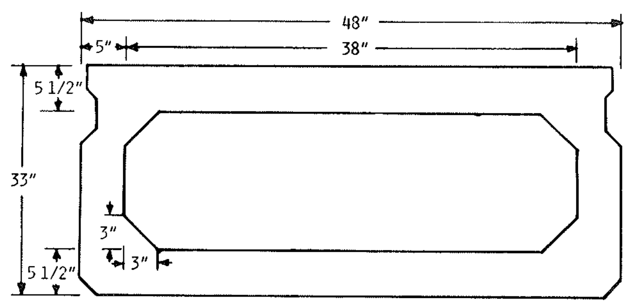

FIGURE 9 Box beam for 60-ft span.

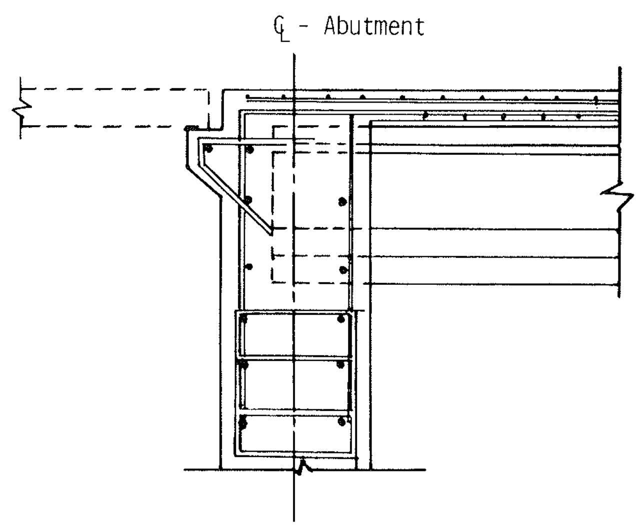

FIGURE 10 Integral abutment.

method results in shorter span lengths but requires the construction of abutments and wingwalls. A piling cap is used in stub abutments to support the girders. This piling cap is much smaller in size than the total abutment, but the span length is increased because a fill-slope is incorporated instead of a vertical wall (*4*).

Tokerud reported that the stub abutment is generally the most economical abutment unless, for example, the use of two spans is required instead of a single span (*4*). Therefore, it should be noted that the savings in girder size will not generally offset the increased substructure cost.

As was previously stated, the concept of prefabrication is applicable to substructures of low-volume bridges. GangaRao proposed 10 prefabricated substructural systems; however, their use has been limited to date primarily because of the unique site variables, such as soil bearing values and depth or location of bedrock (*21*).

Perhaps the most critical question to be answered involves the connection between a prefabricated superstructure and a prefabricated substructure. Because monolithic (rigid-frame) construction is encouraged when it is feasible, this connection (Figure 10) must provide either full moment transfer or at least prevent the translation of the girders over the abutment.

Deep foundations such as piles were not considered because they are probably uneconomical for low-volume bridges. It would be more effective to use a larger, shallow foundation than to use a deep foundation in the remote regions in which low-volume bridges are typically found.

Economic Survey

Several factors affect the determination of the initial cost, including availability of materials, availability of forms and equipment to fabricate and handle one type of element, the qualifications and experience of the available labor force, and the characteristics desired in the finished bridge. Therefore, stock items should be used whenever possible and construction techniques should be employed that are within the capability of locally available equipment and labor crews. Some of these aspects have been described by Sprinkle (*20*).

COST-REDUCTION SCENARIO

A cost-reducing design scenario has been developed, the purpose of which is to outline a series of design decisions that will illustrate the potential savings to be realized if the proposed recommendations are enacted.

For this particular scenario, a traditional 60-ft, simple span, two-lane (34-ft) bridge has been designed with four steel stringers spaced at 8 ft, 4 in centers, and overhangs of 4 ft, 6 in and a 7-in-thick concrete deck. In addition to the traditional design (according to AASHTO specifications), two other alternatives are considered for the sake of comparison: construction of a one-lane bridge and rigid-frame construction. The weight reductions of girders are given in Table 1. Percentage reductions in components are given in Table 2.

CONCLUSIONS AND RECOMMENDATIONS

Two crucial points have been made in this study thus far. First, only limited work has been performed that relates specifically to low-volume bridge design, construction, maintenance, and rehabilitation. Second, potential savings can be realized through reduction in bridge parts, design modification, and effective rehabilitation schemes. The following areas of research may lead to potential savings.

Considering the problems that bridges currently develop as a result of fatigue cracking, it is essential that the unique characteristics of low-volume bridges in regard to fatigue be understood before current AASHTO specifications are changed (*2*).

Many dynamic loading tests have shown that an impact load of 30 percent can be reached at speeds as low as 15 mph. Therefore, it appears that no reduction of the current AASHTO specifications if justified. The type of construction material used should be investigated. For example, timber is less susceptible to impact than other materials because of its excellent energy-absorbing characteristics under dynamic loading situations. Special consideration must also be given to relaxing the deflection requirements.

TABLE 1 WEIGHT REDUCTION OF GIRDERS

Alternative	Girder Size (in)	Weight (lb/ft)	Percentage of Reduction in Weight From AASHTO
AASHTO	36 × 194	194	0
One-lane	36 × 170	170	12.4
Rigid frame	33 × 130	130	33.0

TABLE 2 REDUCTION IN COMPONENTS

Component	Two-Lane Quantity	One-Lane Quantity	Percentage of Reduction of Two-Lane Bridge
Slab	60 ft long × 32 ft wide = 1,920 ft^2	60 ft long × 16 ft wide = 960 ft^2	50
Steel stringers	194 lbs ft × 60 ft × 4 girders = 46.56 kips	170 lbs ft × 60 ft × 2 girders = 20.40 kips	44
Abutments	32 ft wide	16 ft wide	50
Railings	60 ft × 2 = 120 ft	No guardrails	0

Preliminary investigations revealed that many government agencies have used guardrails that are below AASHTO standards on their low-volume bridges. The issue that needs to be resolved is whether or not guardrails are necessary on low-volume bridges; if so, the most economical systems should be designed. Legal aspects such as liability must be considered. Case histories should be studied to establish proper precedents.

A more detailed investigation of the cost of constructing jointless, rigid-frame bridges in remote locations is needed. The ratio of additional costs, such as tying the superstructure down to the substructure, to the realized savings, such as eliminating joints, bearings, and smaller stringers, needs to be defined.

The availability of accurate cost data is essential to evaluate various alternatives with a value engineering approach. This is being performed as a continuation of this study.

An expanded list of design scenarios should be developed with span lengths that range from 50 to 100 ft, abutment heights from 8 to 20 ft, and a soil bearing capacity from 1 to 5 ksf.

ACKNOWLEDGMENTS

The support of the Federal Highway Administration through the Low-Volume Bridge Project (DTF H 61-85-C-00113) to perform this research is gratefully acknowledged.

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The contents of this paper reflect the views of the authors based on their interpretation of the research data. The conclusions of this study should not be regarded as specifications or standards for the design of low-volume road bridges.

Timber Bridges: Part of the Solution for Rural America

ROBERT BRUNGRABER, RICHARD GUTKOWSKI, WILLIAM KINDYA, AND RUTH MCWILLIAMS

A detailed inventory of the condition of highway bridges in the United States has been prepared in recent years. The study described in this paper indicates that an overwhelming proportion of rural highway bridges are on roads that serve low volumes of traffic. As a result of recent bridge failures and the vast number of bridges whose intended service lives have been exceeded, significant federal funding has been targeted for rehabilitation and replacement. The importance of directing an optimal proportion of funds to rural bridges is examined. The poor condition of bridges in rural regions and the impact of the problem on the the rural livelihood and economy is documented. The findings of a search of the National Bridge Inventory to assess the performance and current condition of timber bridges are reported. The function that the use of contemporary timber bridges can serve in addressing the severe rural bridge restoration needs has been identified. Descriptions are provided of favorable factors that were found to pertain to both existing and recently developed timber bridge technologies. These factors provide an incentive to the continued and increased use of timber bridges. A case study in Pennsylvania is documented to profile the nature of timber bridge use and the negative impact of unattended bridge repair needs in a state with a diverse rural economy. Constraints and reservations that have existed in regard to the recent use of timber bridges in rural regions are discussed. An exhaustive program of engineering development, research, and transfer of technology that is related to a plan to significantly increase the use of timber bridges in the rural highway environment is summarized.

RURAL AMERICA AND THE NATIONAL BRIDGE PROBLEM

The deterioration of the nation's bridges is a national problem that affects rural America. Transportation accessibility, which includes time, cost, convenience, dependability, and safety factors, influences rural America's economic activity and development potential.

The Rural Economy

The rural economy has currently diversified beyond its traditional agricultural base (*1*). These changes include a reversal during the 1970s of the decades-long trend of net outmigration; increasing employment from the trade, services, and manufacturing sectors; and less isolation, partly as a result of improved communications, transportation, and employment opportunities. Seven types of rural counties have been identified that characterize about 85 percent of the nation's 2,443 rural counties. The different types of rural activities and their general locations are shown in the map in Figure 1. The farming-dependent counties still compose the largest single group; they represent 29 percent of all nonmetropolitan counties, but only 13 percent of the nonmetropolitan population.

As the rural economy has become more diverse, the options for transporting people and goods have lessened. Rural regions have been especially affected by the abandonment of low-density railroad branch lines and the mergers of railroads. Combined with reductions in air service to rural regions, this has made much of the rural economy dependent on the availability of truck transportation.

In the farm sector, an increase in farm size, mechanization, and productivity has resulted from the use of larger trucks and farm implements, and heavy production inputs, such as feed, fertilizer, and fuel, which are supplied by the nonfarm sector. As the abandonment of railroad branch lines continues, the use of heavy trucks in rural regions can be expected to increase in order to carry production from the farms and other businesses to the distant elevators, ports, and markets. Farm families are also currently relying on the road system to travel to jobs off the farm. In fact, almost 54 percent of the total income of farm households in 1984 came from nonfarm sources (*2*).

In the face of this dependence on trucking, inadequate roads and bridges have resulted in detours and other travel inconveniences, slower delivery times, increased loss and damage of product, higher vehicle operating costs, and generally less efficient transport. Efficiency of transport is a key concern of businesses that serve farms and other centers of rural production because it affects the profitability of their operations and their willingness to serve rural regions.

Rural people and those serving rural regions need and expect the road system to meet their logistical needs. In a 1982 study of the economic and social impacts of deficient bridges, Wilbur Smith and Associates found that the use and delivery of seven vital community facilities and services (e.g., the use of schools and fire protection) were adversely affected by the existence of inadequate and collapsed or closed bridges (*3*).

National Bridge Problem

In its Seventh Annual Report to Congress on the status of the nation's bridges, the U.S. Department of Transportation (DOT) reported that 574,729 highway bridges were inventoried in the United States at the end of 1985 that were 20 ft or more in

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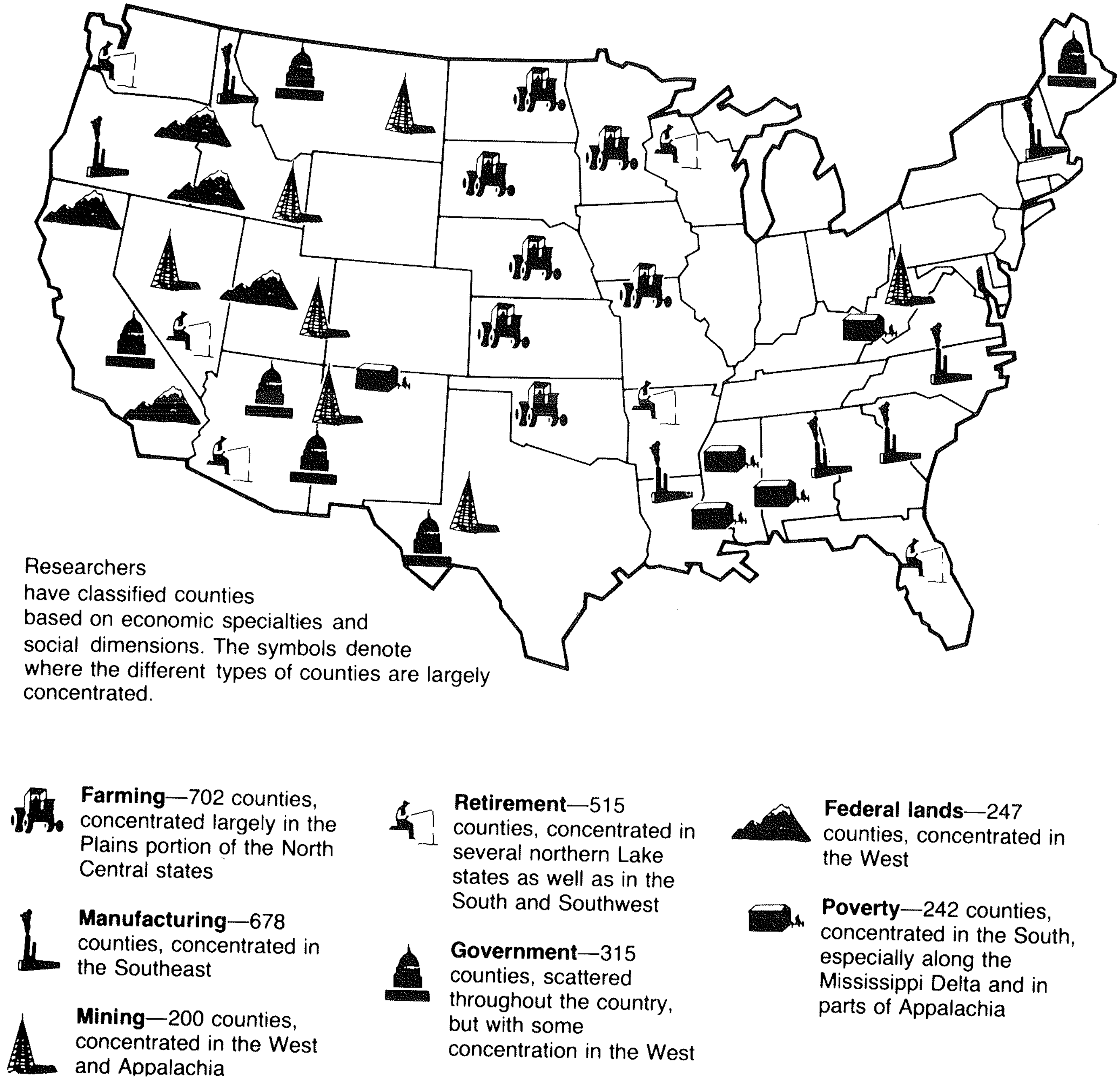

FIGURE 1 Types of rural counties in the United States. Source: *Farmline,* U.S. Department of Agriculture, Sept.-Oct. 1985.

length (*4*). Based on data furnished by the states, the FHWA estimates that about 42 percent of these bridges are deficient for either structural or functional reasons. The distribution of deficient bridges indicates that the bridge problem is national in scope.

Each inventoried bridge is assigned a sufficiency rating to be used to rank bridge reconstruction needs. The sufficiency rating reflects three considerations in the following relative percentages: structural adequacy and safety—55 percent, serviceability and functional obsolescence—30 percent, and essentiality for public use—15 percent.

Based on assigned sufficiency ratings, these bridges are designated as eligible for either replacement or rehabilitation under the Federal Highway Bridge Replacement and Rehabilitation Program (HBRRP). Higher priorities for federal replacement or rehabilitation funds are given to bridges that have lower sufficiency ratings. Bridges that are rated 50 to 80 are eligible for rehabilitation. Those rated below 50 are eligible for replacement. Bridges must be 20 ft or more in length to qualify for either category. Lists of eligible bridges are compiled by each state. Minimum and maximum limits exist for the percentage of annual federal funds any state may obtain. Legislation also directs that the states must use no less than 15 percent and no more than 35 percent of the HBRRP funds each year on bridges off the federal-aid systems. From fiscal years 1979 to 1985, the states obligated 19.7 percent of the total apportioned funds for the off-system (primarily rural) bridges (*4*).

Data published by the FHWA indicate that about 470,000 inventoried rural bridges existed in 1984, which included some bridges that did not receive federal aid (*5*). These rural bridges represent about 80 percent of the nation's total bridges. Almost half of the rural bridges are classified as local in function. A majority of the local rural bridges have relatively low traffic volumes of less than 100 vehicles per day (vpd) (*6*).

About half of the rural bridges also are considered to be deficient for either structural or functional reasons. Many were built in the late 1800s and early 1900s, when traffic volumes were less than they are today and the loads transported over the nation's highways were much lighter. Today's higher traffic volumes and heavier vehicles and loads require wider and stronger bridges. Some of the principal reasons for bridge deficiencies include a lack of proper maintenance that is partly a result of insufficient maintenance funds; hostile environmental conditions; wear from usage; inadequate initial design or construction; and exhaustion of the expected service life.

The classification of a bridge as deficient for functional reasons can often be remedied by upgrading it, but this is contingent on economic factors. A bridge that is classified as structurally deficient has the potential of being unsafe. Indeed, national accident statistics indicate there is a major fatality problem on rural roads (*7*). In 1984, about one-third of the total number of fatal and nonfatal traffic accidents occurred on rural roads. These accidents accounted for over half of all traffic fatalities. The FHWA reports that the most common locations for a fatal accident on a rural local road are at intersections, curves, or bridges, in that order (*8*).

The National Safety Council has explained that "motorist expectancy," based on driving experience and training, is a significant underlying reason for fatal accidents in rural regions, where traffic volumes tend to be low (*9*). The rural road system is traveled by motorists from all parts of the country. For economic and social purposes, it often functions as part of the larger national system. If the road system does not meet driver expectations, then accidents and fatalities are more likely to occur.

Because a deficient bridge is not necessarily unsafe, many of these bridges may be able to safely serve most traffic with proper posting, erection of signs or signals, installation of crash protection, and law enforcement. Unfortunately, such actions may inhibit the movement of heavy trucks and wide equipment, and result in costly detours for these vehicles.

In a study that investigated the economics of reducing the county road system in Iowa, the travel costs for different types of rural travel were estimated for three varying regions of the state (*10*). The majority of the travel in the study areas was made for household purposes. Farm travel, which ranked second, included all farm-related traffic by automobile, farm implements, farmer-owned trucks, and commercial vehicles. Although household travel was a very large percentage of the total miles traveled, it represented a relatively small percentage of the total vehicle travel costs. The cost of farm-related traffic was found to be high in relation to the total farm miles driven.

The rural road system is almost entirely the responsibility of the state and local levels of government. About 66 percent of the rural road mileage is currently under local control; 25 percent is under state control, but this varies considerably by state (*5*). For instance, nearly one-third of the rural mileage in the western states is under federal control because large tracts of land are in federal parks, forests, and reservations. In the South Atlantic region, several states have assumed the legal responsibility for all or nearly all of the local roads. Conversely, in many states of the North Central region, local governments have jurisdiction over most of the rural roadways.

Federal highway funding for rural regions is very limited in comparison to the dollar amount needed to make improvements; therefore, rural officials have relied heavily on other revenue sources. Depending on local and state government structures and laws, a combination of local real estate or property taxes, user fees, and direct federal and state grants, including revenue sharing, has been used (*11*). Rural governments are currently experiencing a high degree of fiscal stress, partly because they must provide costly services to lightly populated areas (*12*). They are also increasingly being asked to serve diversified needs that would further strain their limited resources.

Solutions to the rural bridge problem will vary by state and locality. One possible but previously overlooked solution is the timber bridge. Objective, recently garnered data on the timber bridges in the FHWA's inventory of bridges indicate that timber bridges are numerous, durable, and widespread. Modern advances in timber use have made the timber bridge an increasingly attractive option for small, local road crews.

STATUS OF THE NATION'S TIMBER BRIDGES

Timber Bridges on Public Highways

In 1985, the Technical Committee on Timber Bridges of the American Society of Civil Engineers (ASCE) sponsored a search of the FHWA's National Bridge Inventory (NBI), which includes data on the nation's inventoried public highway bridges that are 20 ft or more in length. The inventory was prepared "to provide Congress with an accurate report of the number and state of the Nation's bridges, in order to guide future legislation in the matter" (*13*). The ASCE Committee sought to produce objective, realistic information on the collective structural status of timber bridges in the United States; to make assessments of their longevity and durability; and to investigate their importance to secondary and rural networks.

The ASCE search was performed in two stages. The first stage was a simple, state-by-state, one-line listing of 11 details about each timber bridge in the bank. The second stage of the search was more exhaustive. Comprehensive matrices of data were produced for each state and then were compiled nationwide. These matrices were used to isolate the past and future performances, a profile of the timber bridge types and lengths, and their statuses. Age, jurisdiction, average daily traffic, and type were each compared with bridge length, estimated remaining life, and status.

In both stages of the search, all data items requested were not available for each bridge listed in the inventory. This resulted in some differences in the total numbers of bridges presented, but did not substantially alter the outcome.

Nearly 65,000 timber bridges in the NBI had main spans constructed of timber as of October 1985 (*14*). These bridges represent about 11 percent of all inventoried bridges on public roads in the United States. The vast majority (over 60,000) of the inventoried bridges can be characterized as longitudinal stringers covered with a deck. Two thousand are described as "deck bridges," which implies that no girders were used. Another 2,000 are called trusses or "frames." Only a mere handful (200) are classified as arches and "other." It is reasonable to assume, furthermore, that the majority of the older public bridges under 20 ft in length is constructed of timber.

Earlier and later inventory data obtained from the FHWA indicate that the number of timber bridges in the United States is declining. In 1983, the NBI included over 70,000 bridges with timber main spans; by September 1986, the number had dropped to just over 60,500 (*6, 15*). Other materials are evidently chosen for replacement and repair projects. Reasons offered for this include prior experience with old timber bridges and lack of knowledge about new timber bridge technologies and their construction advantages, durability, and economic competitiveness.

The ASCE search indicates that about 87 percent of the inventoried timber bridges are located in 19 states. Nearly two-thirds are located in the nine states of Alabama, Arkansas, Iowa, Kansas, Louisiana, Mississippi, Nebraska, Oklahoma, and Texas. In these nine states, timber bridges account for about 20 percent of the inventoried bridges. A high concentration of timber bridges is evident in the central and south central United States (see Table 1). Most of these bridges are located off the federal-aid systems of highways and are classified as deficient (*15*). Most are classified as deficient for structural reasons.

The timber bridges included in the NBI predominantly serve low-volume traffic. Seventy percent of the timber bridges are found on roads that have an average daily traffic (ADT) of less than 100. About one-third of these bridges carry fewer than 26 vpd, one-third carry between 26 and 50 vpd, and one-third carry 51 to 100 vpd (*6*). Of the low-volume groups, 59 percent are short bridges that are 20 to 40 ft long. Only 10 percent of all the timber bridges carry more than 500 vpd.

More than 46,000 of the timber bridges, or 83 percent of the total inventory, fall within the jurisdiction of county and city governments. These governments are primarily responsible for the shorter spans (88 percent of the 40 ft and under spans).

About half (55 percent) of the total of the inventoried timber bridges are open and unposted for any load limitations. However, 1,000 bridges are closed to traffic. Nearly all of the closed bridges are the responsibility of local governments. Forty percent, or nearly 25,000 of the timber bridges, are posted with a maximum legal load limit.

The year of construction of a given timber bridge in the inventory was cross-referenced with the estimated remaining life information to enable an appropriate design life to be approximated for the timber bridges currently in service. The sum of a bridge's age and estimated remaining life would be an indication of its design life. Weighting these discrete design lives by the number of spans falling in a given category yields the average design life of the entire inventory as follows.

$$\text{Average design life} = \frac{(\text{Average age} + \text{average remaining life})_{group} \times n_{group}}{\text{Sum of } n_{group}}$$

The calculated expected life of timber bridges currently in service is 47.4 yrs. This confirms the industry's belief that new timber highway bridges are expected to last 50 yrs (*16*). This is remarkable for older bridges, when one considers how many timber bridges are fairly far along in their life and were probably built with untreated wood. The expected design life results were grouped into design life categories, as can be seen in Table 2.

TABLE 1 NUMBER AND PERCENTAGE OF BRIDGES WITH MAIN SPANS OF TIMBER BY STATE IN 1985 (*4, 14*)

State	Number of Timber Bridges	Percent Timber	State	Number of Timber Bridges	Percent Timber
Alabama	3,171	20.6	Montana	1,829	37.3
Alaska	238	29.0	Nebraska	3,635	22.6
Arizona	109	2.0	Nevada	59	5.7
Arkansas	4,338	33.0	New Hampshire	157	6.2
California	1,276	5.7	New Jersey	289	4.9
Colorado	1,449	20.0	New Mexico	379	11.0
Connecticut	37	0.9	New York	246	1.4
Delaware	61	8.3	North Carolina	2,060	13.0
Florida	838	8.3	North Dakota	1,156	21.3
Georgia	1,196	8.4	Ohio	220	0.7
Hawaii	60	5.7	Oklahoma	3,880	17.0
Idaho	444	11.9	Oregon	1,282	19.5
Illinois	255	1.0	Pennsylvania	342	1.5
Indiana	291	1.3	Rhode Island	17	2.4
Iowa	4,812	18.4	South Carolina	769	8.6
Kansas	2,952	19.6	South Dakota	985	13.9
Kentucky	290	2.3	Tennessee	1,675	9.1
Louisiana	5,924	42.1	Texas	5,712	13.0
Maine	72	2.7	Utah	242	9.9
Maryland	237	5.4	Vermont	90	3.3
Massachusetts	159	3.2	Virginia	110	0.8
Michigan	421	4.0	Washington	1,098	16.1
Minnesota	1,994	15.4	West Virginia	86	1.3
Mississippi	5,920	35.3	Wisconsin	493	3.8
Missouri	712	3.0	Wyoming	449	15.8

TABLE 2 EXPECTED SERVICE LIFE OF EXISTING TIMBER BRIDGES

Expected Service Life	Number of Timber Bridges	Percent of Total
Less than 30 yrs	5,857	11
30-50 yrs	17,891	35
51-70 yrs	22,924	45
71-90 yrs	3,229	6
More than 90 yrs	1,389	3

Over 27,000 timber bridges have lasted longer than 50 yrs. Of these, over 4,500 have exceeded a 70-yr expected service life. About 100 of the bridges have an expected service life of at least 120 yrs. By contrast, almost 6,000 fell within the "under 30 yr" category. This group of spans certainly merits further investigation to determine if a common reason exists for these short design lives. This reason could be determined by tracing the individual bridges in the complete listing; site visits, however, might be the only way to isolate the real causes.

Timber Bridges in the National Forests

The NBI does not include the several thousand bridges that are located in the National Forests and other federal lands that also serve economically important rural traffic. Information about the timber bridges in the National Forests that are maintained by the USDA's Forest Service (FS) was reported in 1984 and 1985 (*17, 18*). Over 11,000 road bridges are maintained by the FS and 100 to 250 are added to the system each year. All-timber superstructures are present in about 55 percent of the FS bridges. Age data are not compiled. In six of the nine FS regions, contemporary glued-laminated (glulam) bridges comprise less than 5 percent of the FS inventory. The percentage is much higher in the other three regions, and ranges from 20 to 75 percent of the total.

The FS replaces or repairs about 270 bridges annually and is seeking methods to both maximize the service life of their existing and new bridges, and still rely on timber as a major material. Field inspection and evaluation of in-place bridges and examination of the FS's national maintenance, rehabilitation, and repair (MRR) needs are key directives in the process.

In 1983, the FS conducted a unique comparative study of 18 experimental timber bridges in the National Forests to assess the in-place performance of the bridges and the merits of dry-use versus wet-use design stresses (*19, 20*). The bridges, located in seven states, were constructed in the late 1960s and early 1970s. They varied in length from 20 to 168 ft, and had 20- to 73-ft span lengths. They were primarily constructed, or re-constructed, with newly developed (at that time) transverse glulam deck panels and a variety of interpanel connections. Some bridges had existing or newly installed nail-laminated decks for comparative purposes. Different types of members, construction, and materials were used in the remainder of the superstructures and substructures.

The inspected bridges were in excellent overall structural condition. Roadway conditions were typically excellent and provided for a smooth passage regardless of surfacing. Extensive asphalt cracking existed only where the surface was unusually thin. Evidence of deterioration as a result of either propagation of cracks or presence of potholes was rare. In addition, the moisture content data obtained from about 100 readings/bridge indicated that the components of the bridges being studied remained below the critical fiber saturation point.

During the summer of 1984, the feasibility of rehabilitating transverse nail-laminated timber decks also was investigated by interviewing FS bridge engineers in the nine FS regions and reviewing available technical literature. The survey revealed the extent of the bridges' needs and current practices and constraints (*17*). The FS bridge engineers generally consider the timber bridge service life to be fulfilled if 30 yrs of use are realized. Rotted deck laminations, excessive maintenance needs, loss of tightness, impaired load distribution, delamination, and asphalt deterioration are major reasons for electing to replace, the first two being the most compelling.

An important contributor to timber bridge deterioration in the National Forests is an inability to implement needed, regular, and thorough maintenance. A low priority is typically placed on maintenance in the FS as a result of limited budget funds. The bridges are seldom rehabilitated (*17*). Rural officials responsible for public bridges also face these decisions. Many of the timber bridges that serve rural regions consequently suffer avoidable degradation. This reality contributes a negative skew to inventory data on condition and estimated remaining life. Both of these facets would exhibit better outcomes with proper maintenance. Conversely, low traffic volumes are favorable to longevity but, in many cases, loads of a high magnitude or impact counter the gain.

The following four national MRR initiatives were recommended to the FS following the investigation; subsequent action has taken place (*18*).

- Computerization of the bridge inventory to put statistics on bridge condition in a common format, and identify and clarify needs;
- Workshops to disseminate information to administrators, engineers, and maintenance personnel;
- Demonstration projects to display and evaluate new methods and technologies; and
- Development of a long-term program to upgrade timber bridges.

INCENTIVES TO USE TIMBER BRIDGES

Advantages

The advantages of using timber for modern highway bridges have been well-documented by suppliers as well as buyers (*21, 22*). These advantages can be categorized into logistical, performance, and economic factors.

The logistics of installing timber bridges are simplified by the material properties of wood. The standard prefabricated glulam panels are lighter than precast concrete panels and steel beams, which enables the use of smaller, cheaper, and more common cranes. Wood can be installed under adverse weather conditions. Finally, the construction of timber bridges does not require sophistication; a typical local road crew can install a bridge with moderate supervision.

An attractive performance feature of timber for bridge use is its complete resistance to the deicing salts. Deicing salts have caused significant and surprisingly rapid deterioration of both steel and concrete bridges and components. Properly treated timber is stable and durable under the most severe environmental conditions. Because wood is salt-resistant, the integrity of toppings, coatings, and seals is not nearly as important as it is in concrete and steel spans. This is reflected in reduced maintenance costs. The resulting prolonged design life also decreases life-cycle costs.

The use of timber bridges can also save material costs. These costs are very site-specific and are a strong function of the relative distances to laminating plants, steel fabricators, and precast factories. The cost differential is rarely dramatic in this very competitive arena. However, savings can be reaped in substructure repair. Because timber's dead load is lighter and its connection requirements are simpler, an otherwise unusable pier or abutment is either adequate or economically reparable. Because its dead weight is light, a timber replacement for steel or concrete can support more live load on the same substructure. Lightweight timber deck replacements alone may obviate substructure or main member replacements. Local officials have often opted for complete replacement with culverts whenever major substructure work is involved.

A timber bridge can be less expensive than a steel or concrete span for several reasons. The greatest savings can be achieved when a small community is able to use its own road crews to install a bridge. In contrast to the heavy and sophisticated equipment requirements needed to install concrete and steel spans, a timber bridge can be installed with light cranes and hand tools. Semiskilled workers can accomplish the task, which means that savings can be derived by avoiding the use of independent contractors.

One of the primary reasons that timber bridges have not been used more often is that potential buyers have little exposure to them. Local officials have never considered the use of timber in their bridges, or their knowledge of its use is limited to old technology. The lack of a comprehensive, up-to-date design manual and general specifications for use by people interested in the timber alternative has widened the technology gap.

Fear of litigation, aggravated by the lack of specifications, has also caused hesitation to use timber. In a recent case in Connecticut, all parties involved, including the town engineer, a consulting engineer, and the town councilmen, were interested in installing a post-tensioned longitudinal deck timber bridge. The engineer eventually opted not to recommend the method because no technical documentation was available stating that such a bridge would meet AASHTO loading requirements for 20 yrs, which was the town's expectation.

The possibility of hydraulic restrictions is an important technical concern in regard to timber bridges. Widely spaced stringers can require a deeper cross-section than the equivalent framing would in concrete or steel. With a given deck elevation, this translates into a reduced hydraulic opening, which can be a problem during a flood. The construction of longitudinal deck bridges is an alternative that can sometimes be used to overcome the problem. The key to the efficiency of the longitudinal deck bridge systems is to distribute the load among the spanning deck pieces. Either transverse spreader beams under the deck or high-strength, post-tensioned steel rods through the deck can be used to distribute the load (*21, 23*).

Much has been written about the nation's infrastructure crisis. The gloomiest predictions would have us imagine a nearly bottomless potential market for replacement bridges. However, a perhaps more realistic approach to estimate the potential market for short-span, rural timber bridges is to speak with people who are currently marketing the product about their hopes and expectations for the future. One such conversation in 1986 yielded the following assessment.

A northwestern design and marketing company has actively pursued the low-volume, rural bridge market with timber bridges for 5 yrs (1981 to 1986). The company has not tried to go nationwide, other than in a few isolated cases, but has concentrated its efforts in Alaska, Washington, and California. This company now sells anywhere from 40 to 60 timber bridges a year. The president of the company forecasts that its volume of the market will rise to as many as 400 a year.

The use of timber has many interesting applications to the rehabilitation of existing spans. Steel girder bridges have been successfully redecked with glulam panels when the original concrete or steel deck was nearly removed by the action of road salts. Old concrete or masonry substructures require little modification to support new timber superstructures. Treated timber pile substructures can perform very well and can support superstructures of any material. Even existing timber bridges can be updated with new timber components. A long story made short on the use of transverse nail-laminated decks follows.

When glulam was first introduced, glulam stringers were placed at larger spacings than corresponding solid sawn stringers. Live load deflection caused the stringers to loosen and moisture to penetrate, and eventually the deck had to be replaced. Glulam deck panels were therefore developed as a replacement for nail-laminated decks.

Cost Aspects

The relative costs of timber, steel, and concrete bridges vary widely and are very site-specific. The following examples illustrate some of the cost considerations involved in making bridge replacements in cases in which the substructures were economically reparable. As with any economic decision made by engineers concerning engineering works, it is important to compare apples with apples and to ensure that all actual and eventual costs are considered.

The town of Canterbury, Connecticut, recently replaced a 27-ft span with a new timber bridge. The material costs alone were $28,000 for the timber, versus $31,000 for steel. These savings are clearly minimal, and not enough to compensate for any significant differences in expected aggravation. The savings available by use of town road crew labor, however, were more substantial. The installed costs of the timber bridge were $40,000, compared to an estimate of $60,000 for precast concrete when the required concrete contractor's costs were included. As a point of interest, had the town elected to qualify for federal funds by meeting AASHTO specifications for the span, the cost would have been nearly $400,000. The town's 20 percent share of that cost would have been $80,000, which means that more money would have been spent for a bridge that they judged to be far greater than the one they needed. This situation is not unlike situations that existed elsewhere in the country, as reported by the U.S. General Accounting Office in 1983 (*24*).

One carefully documented case of a bridge deck replacement in Allegheny County, Pennsylvania, indicates that some dramatic savings were derived by using timber for a new decking material, as shown in Table 3 (*22*).

The dramatic differences in initial costs were influenced by the required addition of a longitudinal steel beam and accompanying abutment widening for the heavier concrete and steel options. Engineers involved in this project foresaw that the

TABLE 3 COST COMPARISON IN 1979 DOLLARS (*22*)

Type of Bridge	Initial Cost ($)	Life Expectancy (Yrs)	50-Yr Replacement Reserve ($)
Glued-laminated timber	50,000	50	50,000
Reinforced concrete	95,000	15	316,700
Open steel grid	105,000	15	350,000
Concrete-filled steel grid	113,000	15	376,700

timber would withstand the action of the heavy salting program of the region for 50 yrs. The expected life for the concrete and steel alternatives was 15 yrs. This figure was based on actual experiences in the region. Significant, long-term cost savings were established for the timber alternative on the basis of this longer design life.

TIMBER BRIDGES AND RURAL ECONOMIC DEVELOPMENT

The FHWA recommends that when states evaluate bridges, set priorities, and select projects on rural roads with low traffic volumes, they should account for sufficiency ratings and other factors that reflect the state's needs, allow local input, and ensure a fair and equitable distribution of funds throughout the state (*25*). This, in part, recognizes that the lack of a complete, effective transportation network can be damaging to a state's economy.

Pennsylvania Case Study

Pennsylvania has a diverse rural economy. Agriculture, forestry, manufacturing, and mining are all important to its economic development. In recognition of the fact that adequate transportation is essential for its rural regions, the Pennsylvania Department of Transportation (PADOT) is examining the highway network in terms that are different than the traditional functional classifications of highways. Priority planning networks have been identified that highlight truck travel and the movement of goods that are important to the state's commerce and industry (*26, 27*). The roadways that do not appear on the networks serve more local purposes.

Four networks have been developed to target limited resources when highway and bridge improvements are considered. They are the Priority Commercial Network (PCN), the Agricultural Access Network (AAN), the Industrial-Commercial Access Network (ICAN), and the Coal Haul Network (CHN). Roadways on the PCN generally carry more than 500 trucks/day or are connector roads for regional industries. The AAN, ICAN, and CHN tend to be lower in traffic volume and consist of roadways that respectively support the state's agricultural and forestry, industrial and commercial, and coal development.

Data on the 1,713 inventoried timber bridges in Pennsylvania were provided by the PADOT's Bureau of Strategic Planning (*28*). Of these timber bridges, 370 are reported to be all-timber and 1,343 to be part-timber in construction. Most timber bridges in Pennsylvania are located in rural regions (91 percent), are 20 ft or more in length (94 percent), are the responsibility of local government (71 percent), and are traveled by fewer than 100 vpd (60 percent) (*14, 28*).

A total of 187 of the timber bridges in Pennsylvania (10.9 percent of the total) are located on a planning network. About 71 percent of all network timber bridges are located on the AAN, which provides access to the agricultural and forestry regions of the state.

A total of 5,237 bridges are on the AAN, 132 of which are timber; 128 of the timber bridges are located in rural regions (*28*). The majority of these bridges is under the control of the PADOT, but the responsibility is shared with the counties, cities, townships, boroughs, and railroads. The PADOT has responsibility for a relatively large share of the rural road mileage (about 30 percent). It is therefore not surprising that 89 of the network's 128 rural bridges are under its control.

The rural timber bridges in the network also tend to serve the higher volumes of traffic present in rural Pennsylvania. Almost 90 percent of these bridges carry over 100 vpd; over 40 percent carry more than 500 vpd. The bridges tend to be relatively long, with a median length of 48.5 ft, but they range in length from 11 to 2,010 ft. Fifty-four bridges are posted with weight restrictions, and another four are closed to traffic. The detour length for the posted and closed timber bridges ranges from 1 to over 99 mi. The median length of a detour around posted bridges is 5.5 mi, and 4 mi for the closed bridges. The detours translate into additional travel times that range in length from a couple of minutes to a couple of hours, depending on the speed of travel. In addition, many of the posted weight limits on bridges may in fact function as closures. The weight limits on the posted bridges range from 2 to 32 tons, with a median limit of 9 tons. Loaded school buses currently weigh between 8 and 10 tons, and commercial supply and agricultural trucks weigh anywhere between 15 and 40 tons, if not more.

The sufficiency ratings provide an insight into the deficiencies of the bridges on the network. Over 80 percent of the AAN bridges have ratings of less than 80 and are therefore eligible for federal rehabilitation funds; over half of these bridges are rated less than 50 and are eligible for replacement funds. The median rating is 72.5 for the open network bridges, 20.8 for posted bridges, and 4.0 for closed bridges.

Implications for Agriculture

As in Pennsylvania, the majority of the timber bridges in the United States is located in rural regions. In fact, the nine states that rank as the greatest users of timber bridges account for about 31 percent of the total cash receipts from agricultural production, 30 percent of the number of farms, and 34 percent of the farm land in the United States (Table 4) (*29*). It is therefore anticipated that the nation's timber bridges are especially important to agriculturally related travel and economic activity.

TECHNOLOGY TRANSFER NEEDS AND ACTIVITIES

It was concluded in discussions between the American Institute of Timber Construction (AITC) and the FS in April 1983 that the use of timber bridges to replace bridges on rural roads could be successfully encouraged with developed timber bridge technology. A fact-finding workshop was subsequently convened in Milwaukee, Wisconsin, in October 1983 to investigate the demand for timber bridges in rural regions and to identify the critical needs for implementing timber bridge technology (*30*). About 40 knowledgeable professionals representing segments of the wood products industry, bridge component manufacturers, the Federal Government, state transportation agencies, university researchers, and professional societies reviewed current capabilities, discussed the incentives and disincentives that exist for the use of timber in bridge construction, and provided advice on needs. The workshop concluded that existing and newly available timber bridge technology could play a vital role in addressing the nation's bridge needs.

Several of the following developments would be important to the optimization of a technology transfer effort:

- The development of a timber railing design that meets the vehicle impact requirements of AASHTO and local officials;
- The development of AASHTO bridge standards that specifically apply to bridges on low-volume roads;
- Synthesized information on the design, construction, rehabilitation, and economics of timber bridges;
- Increased education on wood as a structural material and experience with timber bridges on the part of bridge engineers;
- Documentation of the initial in-place cost and eventual life-cycle economy of timber bridges compared to bridges constructed of other materials;
- More flexible federal highway funding for bridge projects that satisfy AASHTO design requirements; and
- The development of comprehensive standard timber bridge plans to help reduce local engineering costs.

Subsequent to the workshop, the Forest Service, with the help of an implementation team, developed a Technology Transfer Plan for Timber Bridges targeted at state and local highway officials, federal agencies, and engineering and contracting firms involved in specifying, designing, or building highway bridges. The stated goal of the plan is to increase the use of timber bridges 10-fold in 5 yrs. Its objectives include informing the target audience about the advantages of using timber for new and replacement bridges on local and secondary roads, and federally owned property; providing guidance on the rehabilitation of existing timber bridges; and cooperating with mutually interested organizations and associations to improve the nation's road systems by providing safe, economical timber alternatives for bridge replacement needs.

The plan has the following six major components:

- The preparation of a timber bridge design and construction manual,
- The documentation of the cost-effectiveness of timber bridges,
- The development of bridge railing details that meet AASHTO requirements,
- The dissemination of information, particularly at the state and local levels,
- The execution of demonstration projects in the field, and
- The conduct of extensive national publicity activities.

Progress has been made in implementing the plan. The design and construction manual is currently being developed by the Forest Service (*30*). Its contents will include a compilation of existing timber bridge technology; design examples of timber bridges; comparative economics information; maintenance and inspection information; rehabilitation procedures; typical bridge plans; and a bibliography of related information sources. Several sites in the National Forests have been identified for the construction of demonstration bridges using new technologies; the construction costs and the labor and equipment requirements will be documented. A well-attended and highly successful

TABLE 4 TOP-RANKING TIMBER BRIDGE STATES IN 1985 AND THEIR AGRICULTURAL IMPORTANCE (*14, 29*)

State and Rank		Number of Timber Bridges	Farmland (1,000 Acres)	Farm Income ($1000)	Number of Farm-Dependent Counties
Alabama	(8)	3,171	11,500	2,262	6
Arkansas	(5)	4,338	16,000	3,562	29
Iowa	(4)	4,812	33,600	10,087	54
Kansas	(9)	2,952	48,000	6,547	40
Louisiana	(1)	5,924	10,100	1,661	6
Mississippi	(2)	5,920	14,200	2,351	19
Nebraska	(7)	3,635	47,200	7,636	64
Oklahoma	(6)	3,880	33,000	2,906	15
Texas	(3)	5,712	136,300	10,515	61
Total		39,944	349,900	47,527	294
U.S. Total		64,516	1,015,583	151,253	702
Percent of U.S. Total		62	34	31	42

technical workshop was held in Portland, Oregon, in March 1986 to present contemporary design and construction information on engineered timber bridges.

SUMMARY

The state-of-the-art of timber bridge technology in the United States was reported in 1983 (*31*). Since that time, the documentation of national bridge needs and concern about the adequacy of rural bridges to serve rural America have increased interest in the use of timber bridges. In particular, the provision of bridges adequate for the passage of heavy trucks is considered to be vitally important to agriculture and the rural economy.

Rural bridge needs are concentrated in the short- to intermediate-term range. Simple, lightweight, and quickly constructed bridges and bridge components are a priority need and a clear advantage in making needed improvements. The thrust of the renewed technology for timber bridges in recent years is focused on these aspects. Current actions by the timber bridge community are concentrated on establishing and promoting the proven durability and longevity of contemporary timber bridges to ensure their consideration as a viable alternative for making rural bridge improvements. Research and development are under way to provide improved components and novel bridge systems that are economical and simple enough to be used on rural roads.

Every indication is that state and local governments are going to assume a large part of the bridge improvement responsibility over the next few decades. A clear opportunity exists in rural regions for the pronounced use of contemporary timber bridges. The extensive use of all-timber bridges in the national forests has long underscored their capacity for low-volume, heavy-loaded traffic. Ongoing technology transfer efforts are intended to convey this capability to state and local bridge officials, and, by addressing development needs, enable timber bridge elements and structures to be incorporated into their improvement plans.

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Bridge Inspection in Developing Countries

ROBERT BLAKELOCK, JOSEPH M. BARR, AND PAULINE BARR

The United Kingdom Transport and Road Research Laboratory is publishing a two-part guide to bridge inspections. The guide is intended for use in developing countries and provides advice to a district engineer on organizing bridge inspection and record systems, and detailed technical advice to an inspector on the defects he may find while inspecting bridges. A description is provided of the development of the two-part guide, with particular emphasis on the care taken with the communication aspects of the guides. The problems of bridge inspection in developing countries are discussed. The particular need to overcome the shortage of trained engineers is considered and a solution is proposed in which road foremen and other personnel can be trained to perform the majority of inspection work, which would allow the engineers to concentrate on those bridges that require inspection by a specialist. Part One, "A Guide to Bridge Inspection for District Engineers," provides guidance not only on bridge inspection but on record systems; the use of inspection reports in organizing maintenance, rehabilitation, and reconstruction programs; and the preparation and use of a bridge inventory. Part Two, the "Bridge Inspector's Handbook," is a pocket-sized book designed to provide detailed defect-by-defect advice to a bridge inspector. It is presented in such a manner that a road foreman or technician in a developing country who is able to read some basic English should be able to understand.

Large and small bridges form a vital part of a nation's road network. Few people would argue against the need for adequate maintenance, yet bridge maintenance rarely receives the attention it deserves. A system of inspection is needed to organize maintenance. Without regular inspections, it is likely that defects that could be rectified readily and cheaply if caught in time will develop into major problems.

Most developed countries now have formal bridge inspection systems, although in many cases these have only been established on a national basis during the last 20 years. Details of many of these systems can be found in the OECD's *Bridge Inspection 1976* (*1*). However, even in the developed world, many bridge maintenance authorities find it difficult to provide the resources necessary to perform an adequate bridge inspection program, despite the fact that bridge inspection is acknowledged to be cost-effective. In developing countries in which both financial resources and experienced engineers may be in short supply, the difficulties of carrying out a bridge inspection program are often multiplied.

A considerable number of English language publications on the subject of bridge inspection already exist. However, a review of such publications showed that most were written for the specific circumstances of a particular developed country and are therefore unsuitable for use in developing countries in two major respects. First, these books are generally intended for use either by qualified engineers or by personnel who have undergone at least some formal technical education. Thus, the level of technical knowledge assumed may be beyond that which could reasonably be expected of many potential bridge inspectors in a developing country. Second, because most of these publications assume that their audience is capable of reading complex technical English with relative ease, they are often too difficult for people with poor English to understand.

In an attempt to provide suitable guidance on bridge inspection, the UK Transport and Road Research Laboratory (TRRL) is currently completing a two-part bridge inspection guide for use in developing countries. The guide is intended to cover all common small to medium bridges. Large or unusual bridges are excluded, because these need to be inspected by specialist engineers. Part One of this guide is entitled "A Guide to Bridge Inspection for District Engineers." It covers inspection and record or inventory systems appropriate to bridges on low-cost roads, and takes into account the limited resources usually available to manage them. Part Two is the "Bridge Inspector's Handbook," which explains bridge inspection and gives a comprehensive step-by-step guide to inspection, following a set procedure. The handbook contains more detailed but straightforward technical advice on bridge defects than is provided by existing publications. The TRRL has already published a guide for district engineers on road maintenance, the *Overseas Road Note 1: Maintenance Management for District Engineers*, but it deals with bridges in only a very general manner (*2*).

David Brooks and John Parry of TRRL's Overseas Unit were responsible for the guide. This unit has experience in the engineering and operational aspects of transportation in developing countries through its research activities performed on behalf of the British government. The authors of the draft of the guide were from London consulting engineers Rendel, Palmer & Tritton, and from the School of Language Studies at Ealing College of Higher Education. Rendel, Palmer & Tritton is an international consulting firm, founded in 1838, with substantial experience in the design, construction supervision, and inspection of bridges in many developing countries. A language consultant from Ealing College with experience in teaching English to engineers and technicians from developing countries advised on the communication aspects of the guide. Those involved in the production of the guide were therefore able to draw upon a wide range of relevant experience of the developing world.

The basic premise of the new guide is that, in some countries, inspection and maintenance are sadly inadequate, partly because the few trained staff available rarely make visits to their bridges. If trained engineers are unavailable, or unwilling, to inspect the bridge stock, it must be done by others under the supervision of a district engineer. The district engineer, therefore, needs to know not only how to set up and administer a system, but also how to select and train suitable bridge inspectors. Assistance in this task can be found in Part One of the guide.

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Engineering technicians, junior engineers, and road maintenance foremen could be trained to perform bridge inspections. Some road foremen, who are unlikely to have much technical background, are nevertheless felt to be suited to the task because of the practical experience they have gained in their work, and because they are accustomed to working on site, rather than in an office. Part Two of the guide has been prepared so that it can be understood by potential bridge inspectors who have little formal technical training and whose knowledge of written English is basic.

COMMUNICATION

It can be assumed that users of the engineer's guide are relatively sophisticated readers with a reasonable command of technical English. The engineer's guide therefore uses the sort of English one might expect to find in a textbook on the subject, although extra care has been taken to ensure that the language used is clear and unambiguous and that the organization of the whole text is suited to the needs of its users.

In contrast, users of the inspector's handbook are unlikely to be fluent readers of English. Some older inspectors may have attended an English-medium school in which they were able to develop competency in reading and writing English. However, the trend in many developing countries is away from English-medium schools and toward the use of a local language in government schools. This means that many younger inspectors are less likely to have learned to read fluently in English, because their exposure to English will have been more limited. It was therefore assumed that users of the handbook would, in many cases, be slow readers of English texts and would be unlikely to have developed reference skills. The handbook is designed to account for these limitations.

As far as the ability to write is concerned, the Bridge Inspection Report Form has been designed so that the inspector only needs to have a basic writing ability in a language, not necessarily English, that the engineer responsible is willing to accept. In fact, the engineer responsible may prefer his inspectors to write their comments in a common local language, if it exists in written form, rather than attempt to decipher comments written in poor English. However, the amount of writing the inspector will need to do is minimal, because most of the inspection report is completed by placing ticks in the appropriate columns of the preprinted report form.

When the handbook was designed, much thought was given to the ways in which the language could be adjusted to encourage better understanding by the inspector. The approach taken focuses on the likely interests, experience, knowledge, and skills of the target reader, the trainee inspector. For instance, it was assumed that most trainee inspectors would be unaccustomed to learning independently from textbooks, especially ones written in English. It was therefore decided to make the language in the handbook "user-friendly." The text addresses the inspector directly, as would be the case in face-to-face interaction. For example, the handbook does not contain the more standard impersonal forms such as, the fixings must always be checked for looseness and damage." It instead uses the more direct "check to see if the fixings are loose or damaged." Care was taken to ensure that sentences are simple, and that more complex grammatical structures, such as "the scour the river causes can be serious," were avoided. The range of vocabulary has been kept to a minimum and many, though not all, potentially problematic words have also been avoided. Two steps have been taken to cope with vital words that may cause problems. First, the word is introduced in a context that helps to explain the meaning. To this end, much use has been made of labelled diagrams. Second, the handbook contains a glossary that explains those words that may cause a problem. The glossary only gives the meaning each item has in the text. For instance, the entry for the word "fill" only covers the noun form: "FILL (FILL IN FRONT OF THE ABUTMENT)—Soil."

Sometimes areas of conflict naturally arose between the engineers and the language consultant when the text was being prepared. These were eventually all resolved and it is believed that the final text does indeed display the clarity known to be necessary.

Another vital aspect of communication in the handbook concerns visual aspects such as layout and illustration. These are discussed in a later section.

THE ENGINEER'S GUIDE (PART ONE)

Effective bridge management requires that comprehensive records be regularly updated. Advice is given in the engineer's guide on establishing and updating bridge record systems, and a method of inspection is presented that could be undertaken by nonspecialist personnel. Guidance is also given on the use of these data in the planning of bridge maintenance, bridge replacement, and feedback to bridge designers.

Bridge records fall into two categories: unchanging data, such as location and structural details, and changing data, such as inspection reports. The guide recommends a single card record system to record the unchanging data. The recommended card is shown in Figure 1 (front) and Figure 2 (back). The guide contains a detailed description of each data item on the bridge record card with recommendations for a bridge numbering system based on route number and distance to the bridge from the road origin. Advice is given on methods that could be used to establish a new inventory. The use of computers in handling bridge records is also discussed and advice is given on the possible advantages and disadvantages of computer-based systems to supplement the card system.

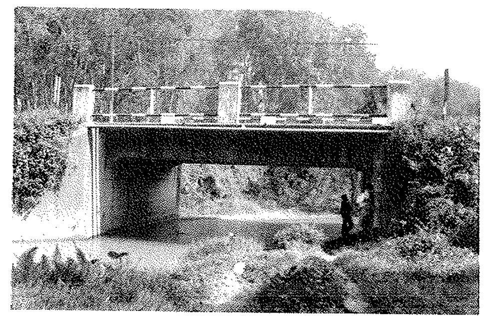
In order to collect changing data, recommendations are made on the management of an inspection program, including frequency of inspection, selection, briefing and equipping of inspectors, and record-keeping. A standard inspection form is used to record this data.

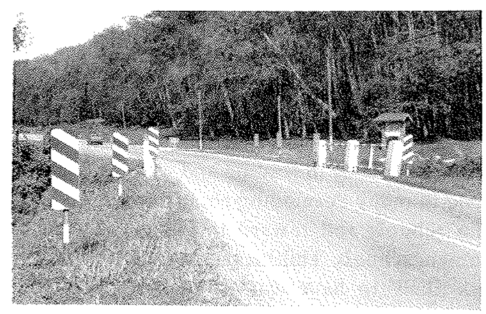
Special attention has been paid to problems that can arise from the use of unqualified staff to perform routine inspections. For instance, because the inspector is likely to have little formal technical education, it is important that the limitations of his role and responsibilities are clearly understood. The engineer's guide describes the inspector as the "eyes of the District Engineer" but cautions:

> "It should be clear to both the District Engineer and his Inspector that inspecting a bridge is a highly responsible job which ideally should be carried out by an Engineer. It should also be clear that it is the District Engineer, not his Inspector, who bears the ultimate responsibility for the inspection. The Inspector should not in general have to make decisions beyond giving his view of the severity and extent of problems he finds".

N 305/2 BRIDGE NUMBER	BRIDGE NAME GOPAL CREEK; ROAD REMBAU - TANJONG	MAINTENANCE AUTHORITY REMBAU DISTRICT - P.W.D.	KILOMETRE 33.6
MAP 1/100,000 - REMBAU		REF 81,536 / 15,036	
ROAD CLASSIFICATION	TRUNK	OVER / ~~UNDER~~	GOPAL CREEK
LENGTH	9.81 m	NAVIGATION RESTRICTIONS	—
WIDTH RESTRICTION	—	HEIGHT RESTRICTION	—
LOAD RESTRICTION	—	ABNORMAL VEHICLE	—
CONSTRUCTION DETAILS			
SPAN(S)	9.5 m		
RUNNING SURFACE			
SUPERSTRUCTURE	R.C. BEAM SLAB		
PIER(S)	—		
ABUTMENTS	R.C. WALL		
FOUNDATION TYPE	SPREAD FOOTING ON ROCK		
ARTICULATION	SIMPLY SUPPORTED		
SERVICES CARRIED	NONE		
DESIGNED BY	WILCOX & PARTNERS		
CONSTRUCTED BY	P.W.D. - REMBAU DISTRICT		
YEAR OF COMPLETION 1965			

PHOTOGRAPHS





LOCATION PLAN

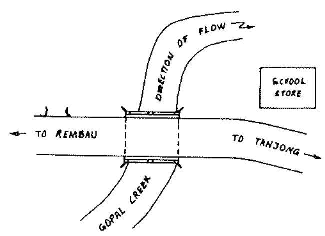


FIGURE 1 Front of proposed bridge record card (A3 size).

Thus, the Inspector reports what he finds but is not expected to make judgments about the implications of what he records as defects. For instance, an inspector may notice and report both that the road surface is cracking just behind the abutment and that the bridge has no room to move on its bearings. However, it is the district engineer who must decide whether these are unconnected events or whether these are signs of forward movement of the abutment. The principle behind this system is that if the engineer does not perform the inspection himself, he must study each of the inspection reports and determine whether a further inspection by an engineer is required.

It is recommended that an engineer perform the first inspection for each bridge. This will not only produce a starting list of defects that require maintenance, but will also ensure that the inspector, on subsequent inspections, will be given the appropriate inspection form with notes covering any aspects specific to the bridge in question.

An appendix to the engineer's guide contains supplementary notes on technical details. These notes expand the basic information presented in the inspector's handbook by more thoroughly treating such topics as cracking in concrete, corrosion bulking of steel, and the special problems of Bailey bridges which receive only a simplified explanation in the handbook.

A further reading list is provided to assist those district engineers who wish to study specific problems or techniques.

THE BRIDGE INSPECTION REPORT FORM

Both the guide and the handbook are based on the Bridge Inspection Report Form, which covers all aspects of bridge inspection for routine small- and medium-sized bridges. Although the form will undoubtedly be of use to a qualified engineer performing inspections, it has been written specifically to be suitable for use by technicians, road foremen, or junior engineers inspecting bridges.

The title page of the Bridge Inspection Report Form (Figure 3) contains all data required to make a positive identification of the structure to be inspected. The need for a clear reference system to elements of the bridge (pier 1, pier 2, etc.) is stressed both in the engineer's guide and the handbook. A sketch plan is provided for this purpose on the title page. Special instructions from the engineer to the inspector appear on this page and there is also space for the inspector to write any urgent notes for the engineer's immediate attention.

The main body of the form is arranged in such a manner that an inexperienced inspector following the order of the form will be led into a logical inspection sequence. This sequence starts with the road approaches and peripheral items that might easily be neglected, such as a missing load limit sign. It then progresses to an examination of the deck surface, the main superstructure elements, and the underside of the deck, including such items as drainage, expansion joints, and bearings. Then the sub-

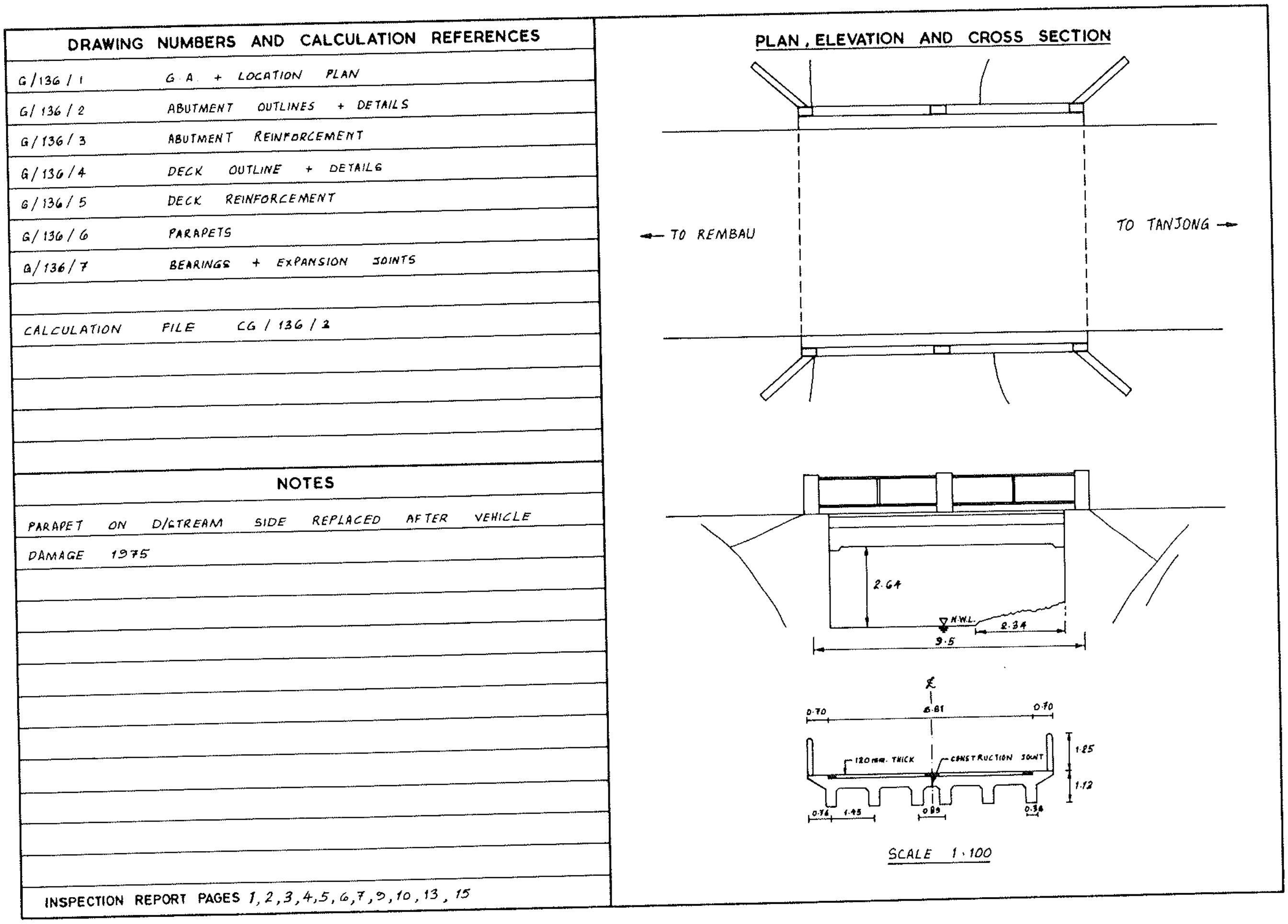


FIGURE 2 Back of proposed bridge record card.

structures, including wingwalls and retaining walls, and approach embankments are reported on. The form includes sections that cover river attack, river training works and inverts, and bed protection. Special sections are included on masonry arches and Bailey bridges.

Different structural types and possible materials are covered in each section of the form through the provision of lists of possible defects, as shown in Figure 4. The inspector is required to consider each possible defect and either cross out the section if it does not apply, or tick either the "No" column if there are no problems, or the "Yes" column if there is a problem. Where a defect is noted, the inspector assesses the severity ("How bad?") and the extent ("How much?") and ticks the appropriate column. The frequently used concept of a condition rating was considered to be beyond the capabilities of most of the inspectors likely to use this report form. Where necessary, the inspector is encouraged to write a note, draw a sketch of the defect, and reference these in the final column of the report form. If the inspector is unclear about how to report a particular defect, column one of the form refers him to the page in the second part of the handbook that deals with that particular defect.

One advantage of using a standard format is that it provides a brief but explicit means of communication between the inspector and the engineer, and therefore reduces the likelihood of confusion. Another advantage is that by requiring the inspector to respond to all the defects listed by placing a tick in one of the two columns, the engineer can be reasonably confident that no defects were inadvertently overlooked. Repeated use of a standard form also enables successive inspection reports to be easily compared, a procedure by which any slow deterioration of elements of the bridge will be made apparent. Finally, the design of the form, in which defects are arranged by material type, simplifies the task of collating the information collected for later use in planning maintenance activities, especially those of a repetitive nature.

It is not intended that the inspector should be given an entire blank report form. Given his limited skills with paperwork, it was believed advisable to provide him with only those parts of the form that are appropriate to the bridge he is going to inspect. It is suggested, therefore, that each bridge record card should list the page numbers of relevance to that bridge so that the blank forms may be prepared in advance by clerical staff (see Figure 1).

THE BRIDGE INSPECTOR'S HANDBOOK (PART TWO)

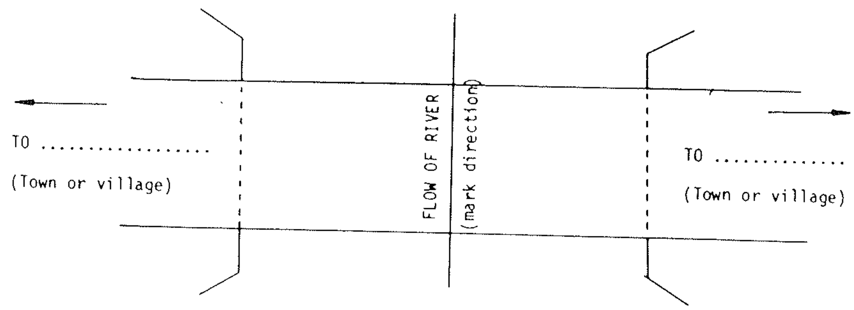
The second part of the guide is intended for use as a reference during inspections, but it is arranged in such a way that it can be used as a training manual. In order to make the book easier to use on site, it was believed that it should be pocket-sized, like the

BRIDGE NUMBER .... NAME ....

CROSSING ....

KILOMETRE .... ON THE .... TO .... ROAD

VIEW OF BRIDGE LOOKING FROM ABOVE



SPECIAL INSTRUCTIONS FROM THE ENGINEER TO THE INSPECTOR

....

....

....

....

SPECIAL NOTES BY THE INSPECTOR

....

....

....

....

DATE OF INSPECTION .... DATE OF LAST

INSPECTED BY .... INSPECTION ....

NUMBER OF PAGES IN REPORT ....
(includes sketches, notes, photos etc)

REPORT ACCEPTED .... ENGINEER

.... DATE

FIGURE 3 Title page of Bridge Inspection Report Form.

UNECA Road Maintenance Handbook, Volume 2, which was prepared by the Overseas Unit of TRRL. In order to make the book as effective as possible, care was taken to ensure that it was attractively presented to encourage trainee inspectors to read it. This was achieved partly through copious use of photographs, color illustrations, and line diagrams. Photographs were used to illustrate particular defects when appropriate, but color illustrations were generally used because they allow some exaggeration, which can be used to focus attention. They also brighten the text and make it appear more attractive, which increases reader motivation. Typical illustrations of the handbook are shown in Figures 5 and 6. Line diagrams were used to illustrate structural actions and typical structural details.

Care was taken to arrange the book in a way that would ease rather than hinder its use by a trainee inspector. Page layout was therefore carefully planned to provide consistency over large parts of the book. Photographs, illustrations, and line diagrams also appear immediately adjacent to the text to which they relate. Because most trainee inspectors are unlikely to have sophisticated reference skills, references to other sections of the book are avoided. The Bridge Inspection Report Form serves as an index to the main text of the handbook. Words used in the form are printed in red in the book to ease cross-reference.

Part One of the handbook begins with a description of the different parts of the bridge, different types of bridges, and different articulation arrangements. Concepts such as movement under temperature change and live load are explained simply. General descriptions of the main causes of damage to bridges are given, followed by an explanation of the defects that beset the different materials used in the bridges. A simple test is described when appropriate. For example, a rivet test is described as follows.

> "You can easily check that rivets are tight and unbroken. Put your finger on one side of the rivet head so that your finger touches both the plate and the rivet head. Then hit the other side of the rivet head firmly with a light hammer. If the rivet is loose or broken your finger will feel the rivet move."

SUPERSTRUCTURE SPAN NO. ______

Handbook Page	SUPERSTRUCTURE SPAN NO. ______ POSSIBLE PROBLEM	NO	YES	HOW BAD? not very bad	HOW BAD? bad	HOW BAD? very serious	HOW MUCH? not much	HOW MUCH? some	HOW MUCH? a lot	Note or Sketch Reference
44	SUPERSTRUCTURE									
45	GENERAL									Not properly checked ☐
	Impact damage to beams, girders, trusses or bracings									
	Debris or vegetation on beams, girders, trusses or bracings or in joints?									
	Water coming through the deck?									
	Water from the deck drainage flowing onto girders, trusses, beams or bracings?									
	Not enough headroom for overbridge?									MIN HEADROOM=.....m
47	MAIN BEAMS, GIRDERS OR TRUSSES									
47	CONCRETE BEAMS OR SLAB									Not properly checked ☐
	Cracking?									
	Spalling?									
	Corrosion of reinforcement?									
	Poor concrete?									
49	STEEL GIRDERS									Not properly checked ☐
	Deterioration of paint or galvanising?									
	Corrosion?									
	Bends in web or flange or stiffeners?									
	Loose bolts or rivets?									
	Cracking?									

FIGURE 4 Sample page of Bridge Inspection Report Form.

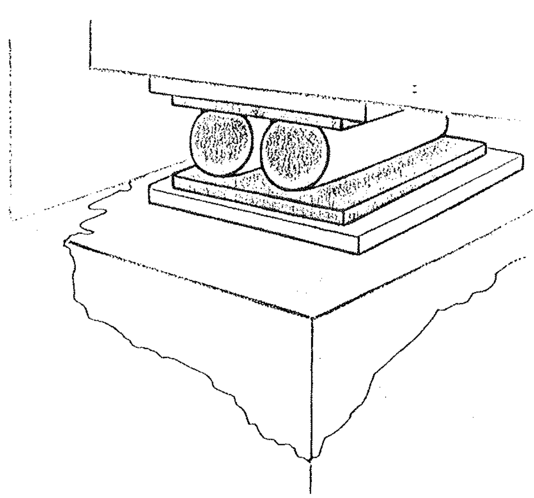


FIGURE 5 Typical handbook illustration: serious cracking near bearing.

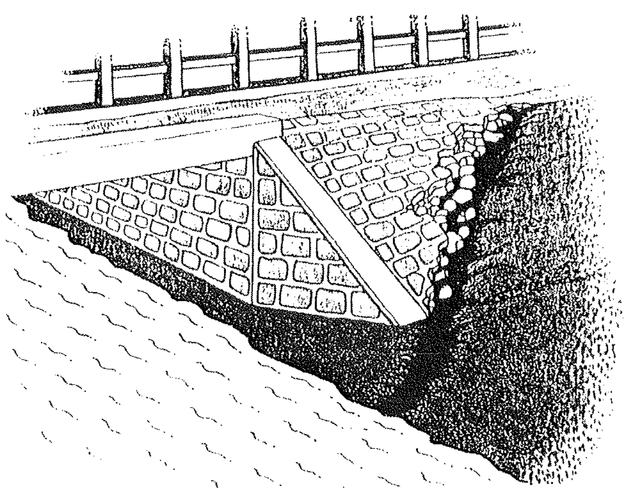


FIGURE 6 Typical handbook illustration: erosion damage to embankment.

Part One thus serves as a basic textbook for all potential inspectors. A road foreman with only a limited knowledge of English may only be able to read one page at a time and it may take a considerable period of time before he can assimilate the information. However, a junior engineer may be able to absorb the contents of Part One in a single sitting. Although the period of time it takes to master the contents of this section is unimportant, it is important that all trainee inspectors understand and learn its contents before they read Part Two.

Part Two precisely follows the order of the Bridge Inspection Report Form and over 200 possible defects are discussed. An explanation is given of what to look for and where to look for it, how to take any necessary measurements, and how to judge such matters as severity and extent. In some cases, only general advice can be given.

> "Brickwork and masonry in a pier may lose some of its motar pointing because of the flow of the river. Later, bricks may be washed out. This can be serious, as the pier is then not so strong. CHECK for poor pointing."

In other cases, the advice is specific. For example a diagram is included that shows where fatigue cracks have been known to occur on Bailey bridges.

Finally, the handbook contains a series of appendices that contain a glossary, a list of recommended equipment, some advice on safety during inspections, an example of a completed Bridge Inspection Report Form, and a blank of the entire form.

TRAINING

It was clear from the beginning that in most cases any training of inspectors necessary would have to be performed by the district engineer. In addition, although guidelines are given in the guide as to the qualities to be expected of an inspector, most district engineers will have a very limited range of possible inspectors to choose from.

The handbook is written to serve the dual purpose of an on-the-job guide and a training manual. Part One in particular is laid out in a manner ideally suited for classroom purposes.

FIELD TRIALS

In order for the guide and handbook to be accepted, it is essential that they are viewed by potential users as practical guides. The authors and the TRRL officers responsible for the guides have been constantly aware of this need. It was therefore decided to subject the guides to field trials before publication. The trials were conducted in Malaysia and Sierra Leone.

The results of the trials might have been considered suspect if a system for the trials was imposed on the responsible engineer. No guidelines other than the guides themselves were therefore issued. In order to ensure some uniformity of response, the responsible engineers were asked to offer opinions on the usefulness and effectiveness of the guides and to answer the following four questions:

1. Are you satisfied that your inspector understands a) the method of inspection, and b) the questionnaire reporting system?
2. Do you have confidence in the data reported in the questionnaire returns?
3. Has the inspection fully revealed the conditions that require attention?
4. Approximately how many hours of instruction were required per inspector before he could be sent out to work alone?

The Malaysian field trials have currently been completed. The results suggest that the guides should indeed be useful and effective. The engineers responsible were satisfied that the inspectors understood both the method of inspection and the questionnaire reporting system.

The inspectors, none of whom had any experience with this type of work, generally had encouragingly few difficulties in understanding and clearly reporting on the defects of the bridge itself. However, real difficulties were encountered in understanding the causes and effects of river damage (scour under structures, aprons, and inverts) and of rainwater run-off (bank erosion and undermining of bankseat abutments). These sections have since been expanded in order to clarify the explanations given. The explanations of one or two technical terms have also been further clarified.

Some minor problems concerning the layout of the report form were revealed. For example, inspectors were unclear about which section to use when reporting on diaphragms and cross-girders. These problems have now been overcome by making small changes to the layout of the form.

As might have been expected, junior engineers took only 1 or 2 days to absorb the contents of the handbook, and technicians took about a week. The trials revealed a tendency for the junior engineers to be a little overconfident. District engineers will also need to ensure that all inspectors follow the guide consistently.

Although it is recognized that other trials may give different results, it was believed that the feedback obtained from Malaysia has been very useful to the authors. It enabled them to redesign or redraft particular points in the guide and report form.

ACKNOWLEDGMENTS

The authors wish to thank Mr. David Brooks and Mr. John Parry of TRRL for their help and cooperation in the preparation of this paper.

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Protection of Wooden Bridge Decks on Aggregate-Surfaced Roads

RICHARD A. FAUROT, DONALD N. MOCKLER, AND ALLAN A. JOHNSON

There are 36 bridges with wooden decks on aggregate-surfaced roads in the Chequamegon National Forest. These decks wear out rapidly, primarily because of the gouging effect of stray pieces of aggregate that are thrown onto the decks by moving traffic. In an attempt to reduce this rapid wear, forest personnel have experimented since 1981 with different applications of asphalt surfacing to protect the decks. Geotextile underlays have also been experimented with in an attempt to reduce the cracking of the asphalt surfacing on the wooden decks. The different applications of these asphalt surfacings and the results to date are discussed.

There are 39 treated timber bridges with wooden decks in the Chequamegon National Forest in northern Wisconsin. All but three of these bridges are on roads that are surfaced with crushed aggregate. The traffic volume on these roads is usually less than 100 vehicles per day (vpd) and consists of light vehicles and logging trucks with a gross vehicle weight (GVW) of up to 40 tons.

The decks of these bridges consist of running plank tracks, full-width running planks, nail-laminated deck planks, or glulam panels. These wooden decks wear out rapidly and are also subject to rot. Some wear results from the friction of tires on the surface, but most wear is caused by stray pieces of crushed rock from the roadway gouging the wood.

Potholes also develop in the road just behind the abutment walls, which creates a maintenance problem and an impact load to the bridges. When an attempt is made to fill these potholes, the graders damage the top abutment planks and crushed surfacing is carried onto the bridges. This material causes the bridge deck to remain damp, which increases the incidence of rot. Moving traffic also throws angular pieces of aggregate onto the bridge, which causes excessive wear.

In an attempt to reduce this rapid wear, Chequamegon National Forest personnel began experimenting with paved approaches in 1981. That year the approaches of two bridges were paved for a distance of 50 ft with cold-mix asphalt. The decks of both bridges had running planks. The situation improved on both bridges, but it was determined that 50 ft was an insufficient pavement length. The graders dragged the crushed aggregate over part of the paving, and pieces of aggregate were still being thrown onto the bridge by vehicles.

In 1982 it was decided to pave three bridge decks that were showing wear. The decks of two of the three bridges were constructed with glulam panels; the other bridge had a nail-laminated deck. Each bridge was designed to have a wearing course, but it had never been applied. Two of the bridges were almost 10 years old and were beginning to show considerable wear.

It was known from experience in other forests that asphalt coatings on glulam decks developed cracks at the panel joints, which created a continuing maintenance problem. A decision was made to experiment with a geotextile underlay to try to mitigate this problem.

A contract was made to pave the three bridges with a hot asphalt mix in 1982. The approaches were also paved for a distance of at least 75 ft. A standard geotextile underlay, Reepav®, manufactured by duPont, was used on one of the panel decks and the nail-laminated deck. A new product called Petrotac® and manufactured by Phillips Fiber Corporation was used on the other panel deck. Petrotac has a preapplied asphalt backing. It is considerably more expensive than standard underlays, but it was hoped that it would provide better results.

An MC30 tack coat was applied at a rate of 0.3 gal/yd^2 on the approaches, and approximately half that rate on the bridge decks, to bond the Reepav to the deck surface. No tack coat was applied on the deck on which the Petrotac was used because the fabric had an asphalt backing. Petrotac was a new product in 1982 and at that time no tack coat was specified. Since then, it has been recommended to apply a tack coat on top of the Petrotac.

The Reepav geotextile was placed on the bridge and extended about 5 ft onto the approaches to ensure continuity between the bridge and the approaches. The geotextile was not used on the remainder of the road.

Petrotac is sold in 1-ft or 3-ft widths in 50-ft rolls. The 3-ft-wide rolls were purchased and placed transversely across the deck over each seam of the deck panels. One strip was also centered on the abutment/fill seam. Because the glulam panels were 43 in wide and the Petrotac strips were 36 in wide, a 7-in space was left between each transverse strip. This space was purposely left bare.

The bridges were 84, 77, and 71 ft long. The bridge with the nail-laminated deck was 77 ft long and the bridge on which the Petrotac was applied was 71 ft long (see Table 1). The two rolls of Reepav cost $493 and the Petrotac cost $583. This amounted to a cost of about \$1.75/yd^2 or approximately one-third the cost of the bituminous surface of \$4.90/yd^2.

The paving contract was made in September of 1982; a record rainfall was experienced in the fall of that year. After the weather cleared and paving was to proceed, it was found that the aggregate road surface was too moist for adequate penetration of a tack coat. Instead of postponing the project until spring, the contractor was allowed to use a cement block sealing compound cut with gasoline as a tack coat to bond the Reepav. This worked well, but probably not as well as a light MC30 tack coat.

The two bridges on which the Reepav fabric was used were paved on the same day. The temperature on that day was

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TABLE 1 BRIDGES AND TREATMENTS

Year	Bridge Name	Length (ft)	Deck Composition	Treatment
1981	Hungry Run (FR 164)	26	Full-width running planks	50-ft approaches
1981	East Fork, Chippewa River (FR 164)	175	Two-track running planks	50-ft approaches
1982	South Fork, Flambeau River (FR 149)	77	Nail-laminated	Reepav, deck, and approaches
1982	South Fork, Flambeau River (FR 152)	84	Glulam panels	Reepav, deck, and approaches
1982	Brunsweiler River (FR 196)	71	Glulam panels	Petrotac, deck, and approaches
1984	Elk River (FR 131)	51	Glulam panels	Reepav, deck, and approaches
1984	Yellow River (FR 121)	93	Nail-laminated	Reepav, deck, and approaches
1984	North Fork, Yellow River (FR 108)	51	Full-width running planks	100-ft approaches
1984	North Fork, Yellow River (FR 112)	58	Full-width running planks	100-ft approaches

between 42° and 50°F (6 and 10°C), which was not an ideal temperature for paving. Shortly after the second lane on the second bridge was started, it began to rain hard for about 15 min. When the rain stopped, the water was swept off the bridge and paving was completed.

It snowed the day after these two bridges were paved; the third bridge could not be paved for 2 weeks. Some ice was on the shoulders of this bridge on the day paving was resumed. The ice was melting when the Petrotac was being applied, which dampened the deck and prevented the Petrotac from bonding before the asphalt was applied. The material tended to gather under the paver and form pleats under the asphalt, particularly at the center of the bridge. The temperature of the hot mix was between 260° and 295°F (127 and 146°C). The heat was expected to evaporate the moisture on the deck and enable the Petrotac to bond. This apparently succeeded because no pleating problems have been observed to date.

In 1983, Nicolet National Forest personnel paved two glulam panel decks with Petrotac by laying it longitudinally across the full width of the deck. The decks and at least 75 ft of the approaches were paved with 2 in of hot asphalt mix. The cost of Petrotac rose dramatically between 1982 and 1983. The bid price for Petrotac was $6.75/yd^2, whereas the cost of the 2-in hot mix was $6.50/yd^2.

In 1984 four more bridges and approaches were paved. One of the bridges had a nail-laminated deck that had been in place for over 12 years. The deck on this bridge was worn as much as an inch in places. It was decided to place a standard geotextile mat on this bridge and pave it with 2 in of hot asphalt. This same treatment was applied to another bridge that had a deck of glulam panels.

Two other bridges had full-width running planks. The approaches of these bridges were paved with the same hot-mix asphalt for a distance of 100 ft.

The bridges and approaches have been monitored each year since they were applied. As was previously mentioned, it was found that the 50-ft approaches were too short, so they were extended. The longer approach length appears to be keeping most of the aggregate particles off the bridges.

The geotextile underlays are not performing as well as anticipated. After 2 or 3 years service cracks began to develop at the panel points on the glulam decks and randomly on the nail-laminated decks. It was hoped that there would be less cracking. These cracks appeared to be narrower than those observed on decks with no underlay; the use of geotextiles therefore cannot yet be considered a total failure. The cracks must be sealed, however, and that represents a maintenance cost.

One of the bridges in the forest is on a paved road and has a deck that consists of full-width running planks. This bridge was constructed in 1973, and the running planks do not show any appreciable wear yet. It appears from this example that another possible solution is to pave the approaches for a considerable distance, possibly 200 ft, to ensure that the aggregate is kept off the deck. The deck can then be replaced when it is worn out.

Materials Characterization of Cold Asphalt Recycling of Rural and Urban Low-Volume Roads

HUMBERTO CASTEDO AND LEONARD E. WOOD

Pavement cores were taken from different county roads and city streets throughout the State of Indiana with the specific purpose of obtaining materials for laboratory testing and evaluation. Detailed information was gained from these analyses on the asphalt content, asphalt penetration and viscosity, aggregate gradation, pavement layer thickness, and other parameters of the existing asphalt pavements. The state was divided into three geographic and climatic regions, and into rural and urban traffic types. Within these categories, samples were taken from various road sections for varying numbers of layers. The data obtained from these experiments were statistically analyzed and the results were examined and discussed in an attempt to characterize these pavements. This procedure was adopted mainly because of the lack of information available on the existing pavement materials, which is a situation that is typically faced by most low-volume road agencies. It was found, among other things, that (a) the variability of asphalt content in rural or urban low-volume roads could be considered small enough that no detrimental effects would be experienced by the final recycled mix; (b) the average variability within a road section is similar for urban and rural asphalt pavements; and (c) the geographical location of the pavement has no significant influence on the oxidation of the various asphalt layers that form those pavements. The significance of these and other factors believed to influence the performance of cold, recycled asphalt pavements was determined from these analyses.

Existing asphalt pavements from low-volume roads are likely to consist of layers of asphalt concrete of different composition and characteristics. It is also likely that a section of pavement selected for recycling can vary in materials composition from one end to the other or one layer to another. These variations could have been the result of normal construction practices, normal maintenance practices, or weather and pavement age variations.

Existing stockpiles of salvaged material could similarly have been obtained from pavements that also have different characteristics. Test data from samples obtained in highly cracked areas can display properties different from test samples taken from uncracked areas of the same asphalt pavement. The variability introduced by these characteristics can be high in many cases. However, some pavements can be relatively uniform throughout the test section being analyzed.

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Pavement cores were taken from different low-volume roads throughout the State of Indiana with the specific purpose of obtaining materials for laboratory testing and evaluation. It was expected that information would be obtained from these analyses that would aid in the creation of a practical and realistic set of guidelines for the recycling of asphalt pavements in Indiana counties and cities. The entire network of county roads and city streets of the state was the inference population of this evaluation.

PAVEMENT SAMPLING AND TESTING PROCEDURES

Pavement Sampling Procedures

A portable drill with a 4-in diameter bit was used to obtain the pavement cores. The locations of the low-volume road pavements sampled were selected on the basis of the following:

- Their geographical distribution in the state (northern, central, or southern);
- Their potential for recycling, such as pavements with extensive deterioration that are in need of immediate repair;
- Their volume and type of traffic, such as farm to market and suburban pavements; and
- Their typical characteristics.

The selected road was considered a representative pavement section of a particular county or city through an initial discussion with the local highway engineer and a visual on-site inspection.

The number of samples collected and the locations and parameters measured were believed to provide a statistically sound representation of these asphalt pavements and the materials of which they were composed. The data obtained from the pavement cores were used in the statistical analysis of variance (ANOVA) method to determine the extent of the existing variability among and within pavements throughout the state, and to identify the main factors responsible for this variability (*1*).

Pavement Testing Procedures

The pavement cores obtained from the locations shown in Figure 1 were subjected to a series of laboratory tests to obtain data for the response variables used in the statistical analyses.

The cores that represented various pavement layers were weighed in air, and the height of each layer was measured and recorded. The cores were sliced along their diameter into

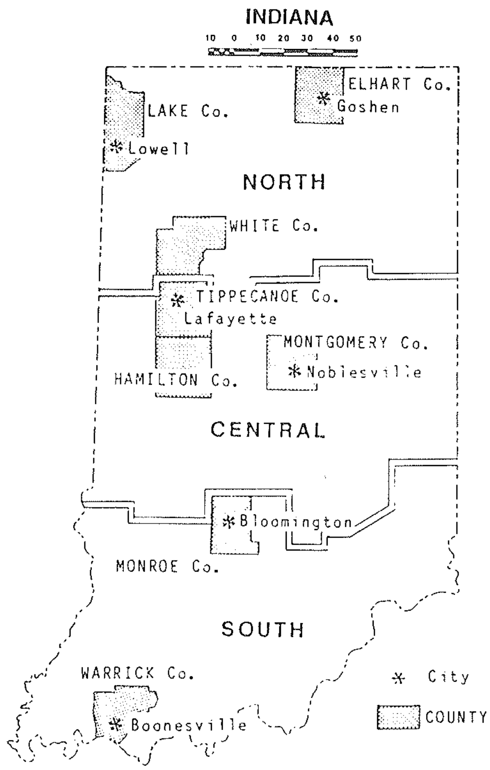

FIGURE 1 Location of low-volume roads.

smaller samples that represented the individual layers that formed each particular pavement core. There were between one and four layers in each core.

The following information was obtained from each sliced layer using standard ASTM test procedures (*2*).

- Specific gravity (density) of the layer was obtained by means of ASTM D 2726.
- Marshall test parameters were obtained following the procedures delineated in ASTM test D 1559. The Marshall stability (lb) and the Marshall flow (0.01 in) were obtained for each layer at room temperature (±72° F) instead of the 140° F that the standard test calls for.
- The asphalt content and the gradation of the recovered aggregate were obtained after the core layer was subjected to extraction procedures given in test ASTM D 2172.
- The sieve analysis of the extracted aggregate was performed using ASTM C 136.
- The recovery of the extracted asphalt was obtained using the test procedures specified in ASTM D 1856.
- The penetration and viscosities of the weathered binders from each layer were determined following standard procedures ASTM D 5 and ASTM D 2170, respectively.

ANALYSIS OF TEST DATA

Statistical Analysis Variables

Six response variables were selected from all the parameters measured for each core layer. These variables were believed to best characterize the weathered and in-place material that formed the low-volume roads analyzed. The response variables were as follows:

- Asphalt content,
- Asphalt penetration (0.1 mm),
- Kinematic viscosity (cSt),
- Aggregate gradation modulus,
- Marshall stiffness, and
- Pavement thickness.

A total of 159 values for the parameters of the recovered original binders was recorded. The asphalt content (percent by total weight of mix) was determined for each separate core layer of the pavement of a county road or city street. A total of 227 asphalt contents were determined.

The aggregate gradation modulus, a measure of the surface area of the extracted aggregate, was used to analyze the existing variability in the original aggregate gradation. A review of the literature revealed that the coarseness of an aggregate gradation can be expressed by a single number that reflects the amount of material passing the 10 standard sieves from 1 1/2 in to No. 200 (*3, 4*). This number can be obtained by adding all the percentages of material passing these sieves for a sample and dividing the sum by 100. This parameter is sufficiently sensitive to reflect changing requirements for mix proportions or asphalt content requirements as the aggregate grading varies (*4*). Because 227 sieve analyses were performed on the extracted aggregate, the same number of observations were made for the aggregate gradation modulus.

The Marshall stiffness of the existing asphalt pavement mixture was obtained by dividing the Marshall stability (lb) by the Marshall flow (0.01 in) of each core layer. The Marshall stiffness (lb/in) has been used in previous studies to evaluate asphalt pavement mixtures properties (*3, 5*). This parameter was believed to better characterize the stability and flow properties of the existing pavement layers than if stability and flow were considered as two, separate test response variables. A total number of 178 Marshall stiffness values were obtained in this part of the study.

The last dependent variable, the thickness of each pavement layer, was measured and recorded. This variable was considered to be of some significance in this study because it is an approximation of the quantity of asphaltic concrete material that can be milled off the existing pavement. A total of 228 layer thicknesses (in inches) were recorded.

Six independent variables or factors were used in this study, as shown in Table 1 and the following list:

- Geographic or climatic region,
- Traffic type (rural and urban),
- Political zone (county and city),
- Roads sampled,
- Pavement core samples, and
- Pavement layer.

The geographic or climatic regions in which the low-volume road asphalt pavements were located were incorporated in the statistical analysis of the data. This was done to determine if there were significant climatic effects on the weathering of the original asphalt binder of pavements located in the northern, central, and southern parts of the state (see Figure 1). The penetration data of the recovered asphalt were used to analyze the effects of somewhat mild weather in southern Indiana against the effects of the most severe winter weather in northern Indiana.

TABLE 1 MAIN FACTORS CONSIDERED

Independent Variables	Sample Size	Description
Climatological regions	3	Northern Indiana Central Indiana Southern Indiana
Traffic type	2	Rural Urban
Zones surveyed	14	8 counties 6 cities
Roads sampled	58	34 county roads 24 city streets
Pavement cores	117	74 from county roads 43 from city streets
Pavement layers	227	114 from county roads 113 from city streets

The political zones (county and city) in which these asphalt pavements were located are also shown in Figure 1. A total of eight different counties and six cities were surveyed in this study, and pavement cores were taken from representative locations.

A total of 34 county roads and 24 city streets were sampled. This factor allowed the existing variation between and within county and city asphalt pavements to be determined by comparing the data obtained for the response variables that were described earlier.

A total of 74 pavement core samples from county roads and 43 samples from city streets were taken. This factor allowed a comparison to be made of the response variables within a given county or city low-volume road. The extent of the variability along the length of a given section of the same asphalt pavement could then be determined.

A total of 144 pavement layers from county roads and 83 layers from city streets were sampled. This factor enabled a determination to be made of the variability between layers from the same asphalt pavement core.

Results of the Statistical Analyses

The analysis of variance (ANOVA) method was used to determine the extent of the materials variability in low-volume road asphalt pavements (*1*). The ANOVA method consisted of mathematical models that incorporated the main factors or independent variables, the use of statistical packaged programs that manipulated the data or response variables, and the careful analysis and interpretation of the output obtained from analyses of these data (*6*). The mathematical and statistical definitions, general models, data layout, and other assumptions used in this experimental design can be found elsewhere (*7*). The ANOVA test results are summarized in Table 2.

Asphalt Content of the Reclaimed Pavement Material

It was found that the average asphalt contents of low-volume county road and city street pavements did not vary significantly. The statistical analysis of the data on this parameter also showed that no significant differences existed between all the roads within a particular county or city. No differences also existed in the asphalt content of all the road sections when they were all considered as a single factor in the analysis. No significant differences existed within a particular road section. It was found that on average, a county road or city street asphalt pavement could be considered to have an asphalt content equal to the average result of a minimum of four asphalt extraction results.

This means that low-volume road asphalt pavements in counties and cities throughout Indiana could be considered to have similar variations in average asphalt content. The same considerations can then be applied to this parameter in the mix design of either rural or urban cold recycling asphalt mixtures throughout the state.

The main differences in asphalt contents existed between the various layers that constituted the pavements. The asphalt content of the upper layer (surface course) was generally found

TABLE 2 SUMMARY OF ANOVA TESTS RESULTS

	Variables					
Source	Asphalt Content	Pene-tration	Viscosity	Gradation Modulus	Marshall Stiffness	Thickness
Traffic Type (TR)						
Zones Z (TR)		*			*	*
Roads R (TR Z)		*	*			*
Samples S (TR Z R)						
Layers (L)	*				*	*
L*TR						
L*Z (TR)	+				*	
L*R (TR Z)	*	*	*	*		*
L*S (TR Z R)						

Notes:
* Statistically significant at α = 0.05;
+ Statistically significant at α = 0.10

to be ±1.0 percent (by total weight of mix) higher than that of the lower layers. The asphalt contents of the lower layers varied within ±0.8 percent (Table 3).

The other main conclusion that can be drawn from the statistical analysis of the data is that the asphalt content within a road section varies significantly from layer to layer vertically and not necessarily within the section and horizontally across the pavement. Each layer of pavement should be milled off separately, and the salvaged material of each layer should be stored or used individually if the variation in asphalt content among layers is found to be larger than ±1.0 percent. The depth of cutting, planing, or milling is an important factor in obtaining a reclaimed material with small variations in asphalt content. Determinations of the pavement layers' thickness should be made as accurately as possible.

Penetration and Viscosity of the Original Binder

The parameters of penetration and viscosity measure the extent to which oxidation and other chemical and weather-related factors have altered the ductility and flexibility of the original asphalt binder. These changes usually result in high viscosity values and low penetrations of the recovered asphalt. The outcome is an asphalt pavement with various degrees and types of distress. Penetration and viscosity values were found to be significantly different from one road pavement to another, and within a particular road or street section, from layer to layer. From the observations of the ANOVA results it was concluded that there were no practical differences of these two parameters among the pavements of the eight counties and six cities analyzed.

It was also found that although these two parameters are weather related, the effects of the various climatic regions found in the State of Indiana (see Figure 1) were minimal or nonexistent. Asphalt binders from the pavements of low-volume county roads or city streets can be considered to age in a similar manner. Finally, the variations in the hardness of the original asphalt within the road section to be rehabilitated seem to be minimal and of small practical importance. However, the variations between layers from pavement to pavement were found to be significant. This means that the various layers of a particular asphalt pavement can be scarified, ripped, or cut and mixed together, and can be considered as having an average asphalt penetration value throughout the entire recycled section for mix design purposes.

These findings are important to the proper design of the recycled mix. Rejuvenating, softening, modifying, or recycling agents are generally used to restore the ductility and the original properties of the asphalt. The optimum quantities and type of recycling agent depend to a large extent on the hardness of this asphalt residue. It can therefore be stated that the same mix design can be used in a recycling project if the salvaged material within a particular pavement section comes from layers with a similar asphalt penetration and viscosity. These two parameters should be carefully determined, and the same type and proportions of recycling agent should be used only for the particular road section for which they were determined.

Gradation of the Extracted Aggregate

Aggregates recovered from existing pavements were not found to differ significantly in gradation. Aggregate gradations from

TABLE 3 DEPENDENT VARIABLE MEAN VALUES FOR ALL PAVEMENT LAYERS

DEPENDENT VARIABLES		PAVEMENT LAYER 1	2	3	4
Asphalt Content	mean:	5.98	4.90	5.05	4.87
	s:	1.42	2.85	3.75	1.03
(%)	n:	117	84	23	3
Penetration of Recov.	mean:	42	40	48	29
	s:	8	10	9.6	22.6
Asphalt (0.1 mm)	n:	78	59	20	2
Kinematic Viscosity	mean:	679.4	750.6	677.0	1137.5
	s:	90.0	616.5	346.4	1055.0
of Recov. Asph. (cSt)	n:	78	59	20	2
Gradation Modulus of	mean:	6.36	6.10	6.29	6.37
	s:	0.03	0.43	1.14	0.86
Extracted Aggregate	n:	117	84	23	3
Marshall Stiffness	mean:	445.51	329.08	352.75	366.05
	s:	52.25	190.97	234.50	249.30
(lb/in. x 10^3)	n:	93	65	18	2
Pavement Layer	mean:	1.33	1.87	1.68	2.08
	s:	0.44	1.41	0.89	0.63
Thickness (in.)	n:	106	79	20	3

Note: n = Number of observations per pavement layer.

s = Standard deviation.

the pavements of low-volume county roads and city streets were found to be similar, as can be seen by the narrow gradation band of the graph shown in Figure 2. The gradation range of a standard, virgin aggregate for base course asphalt mixtures is plotted in this graph for the sake of comparison.

The aggregate gradations were found to differ only between layers of the various road sections considered in this study. A close analysis of the aggregate gradation moduli of these aggregates (see Tables 2 and 3) shows that the differences obtained between average values are not great enough to be of any practical significance. This means that an average value of the gradation (percentage of material passing) of the various layers that are being considered for recycling in a particular project can be used for mix design purposes. These average gradation values would inform the design engineer of the required sizes and proportions of virgin aggregate that the reclaimed pavement material needs to meet a particular standard gradation for base, subbase, or surface course mix.

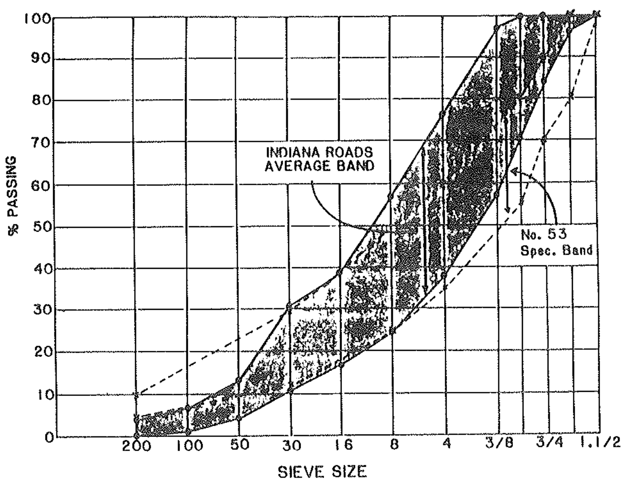

FIGURE 2 Aggregate and standard gradation of average roads.

The aggregate gradation modulus measures to some extent the necessary amounts of binder that a particular aggregate may require to perform well as a paving mix under varying traffic loads and weather conditions (*4, 7*). The statistical analysis of these data showed that the differences in aggregate gradation among pavements of low-volume roads from counties and cities were insignificant. The differences in the gradation moduli between pavement layers showed a statistical significance (Table 3); however, a closer inspection of the mean values of these data indicated that there were no practical differences. The top layers showed a slightly larger gradation modulus (more fines) than the lower layers (see Table 2), but these differences were not great enough to be considered of practical significance.

Marshall Stiffness of the Asphalt Pavement Layers

This parameter measures the relative stability and strength of an asphalt pavement mixture, which in turn relate to the properties of the materials that form a particular mix. The soundness, strength, and size distribution of the aggregate, or interlocking characteristics, influence the stability and stiffness of the mix. The properties of the binder are also reflected in this parameter. Hard, stripped asphalt with no binding properties, and soft, oily asphalt create weak and unstable mixes.

The statistical analysis of the Marshall stiffness data obtained from the core layers of rural and urban road pavement in Indiana led to the following conclusions.

The stability of the mixes that formed the various pavement layers were statistically different from each other, and the top layers were the stiffest of them all in most cases (see Tables 2 and 3). This is because newer, less deteriorated and therefore stronger mixes form the wearing surface of most pavements in service.

The differences in average Marshall stiffness between the county and city pavements of the study were also found to be statistically significant. However, a close analysis of the ANOVA test results indicated that the mean squares of the data for the layers factor was by far the largest of the mean square values. It was therefore concluded that the only practical differences among pavement stiffness occurred among the layers that formed the pavement.

The practical significance of these findings is that if a recycling project is undertaken with the sole objective of restoring the surface or top few inches of a deteriorated asphalt pavement (i.e., surface recycling), the strength and stability of the underlying materials must be carefully determined. If a new, recycled asphalt mix is placed on top of weak and unstable pavement layers, the structural support will be such that the old distresses and faults that made the pavement a candidate for rehabilitation in the first place are likely to be evident again in a short time. More detailed information on cold recycling methods that improve the structural capacity and increase the service life of asphalt pavements can be found elsewhere (*3, 7-9*).

Pavement Layer Thickness

This parameter was analyzed to help characterize the variability that exists in the asphalt pavements of Indiana's secondary roads system. It is a parameter of limited significance for the design of recycled mixtures; however, it helps to measure the relative quantities of existing materials that are available for recycling, and determines the depth of cutting during the reclaiming operation.

The statistical analysis of these data indicated that pavement layer thicknesses were significantly different between the layers of a particular low-volume road pavement, and between the layers of pavements from the various counties and cities investigated in this study. The lower layers of the pavement are thicker, on average, than the upper layers. This is because the lower layers are usually formed of a thick base course and the upper layers are formed of relatively thin surface treatments or asphalt overlays.

It was found that most common routine maintenance practices adopted by county and local highway agencies were temporary measures that consisted of patching, chip seals, and surface treatments with a limited service life and that offered almost no structural support. Heavily traveled roads generally received a hot-mix asphalt overlay when necessary. A breakdown of the type of low-volume road pavements found throughout Indiana was not possible in this study because of a lack of recorded information on the types of materials and construction techniques used to build and maintain most of these roads. Therefore, no attempt was made to determine the significance of the variability that existed between the various types of pavements.

However, it was found from available historical data and from observations of the core samples in the lab that almost all surface layers of city streets were hot-mix asphalt overlays. Most of these layers were built on top of an existing overlay or on top of some kind of cold-mix asphalt material. Lower layers, if any, were generally composed of granular base material or weathered asphalt concrete. However, about half of county roads sampled were found to have a surface layer of hot-mix asphalt concrete in regions in which traffic was heavy. Remote and lightly traveled county roads were found to have a cold-mix (plant-mix or surface treatment) pavement type. The surface layer, which in some cases was as thin as half an inch, was found to be built on top of a granular base course or a weathered cold- or hot-mix asphalt concrete. Sand mixes generally were not found among the pavements analyzed. The subgrade of the pavement was usually composed of existing clayey and sandy soils.

CONCLUSIONS AND RECOMMENDATIONS

An extensive study was conducted to determine the variability of the characteristics and materials that form the low-volume roads of Indiana's counties and cities. The analysis of variance of the data obtained from the properties of the evaluated pavement cores led to the following conclusions.

The pavement samples obtained before milling, ripping, breaking, or other processing occurred were likely to exhibit variable results from the penetration and viscosity of the original asphalt binder to the stiffness of the existing asphalt mix.

The average asphalt content found in county roads and city streets did not vary significantly. This means that the same considerations can be applied to this parameter in the design of either rural or urban cold recycling mixtures. It was also found that the asphalt content was approximately the same for all pavements within a particular county or city, and within a section of road from one end to the other.

The main differences were found among the various layers that constituted the pavements under study. Each layer of pavement should be milled off separately, and the salvaged material of each layer should be stored or used individually if the variation in asphalt content between layers is found to be greater than ± 1.0 percent by weight of total mix. The depth of cutting, planing, or milling is an important factor in obtaining a reclaimed material with small variations in asphalt content. Determinations of the pavement layer's thickness should be made as accurately as possible when large variations exist.

The analysis of the penetration and viscosity data of the recovered original asphalt binders showed similar results. The geographic location or climatic factor of low-volume road pavements does not significantly affect the penetration or viscosity of the existing binders. No practical differences existed between the county or city pavements analyzed. The penetration and viscosity values were found to be significantly different only from one road pavement to another, and within a particular road section, from layer to layer. This appeared to result mainly from local effects that are probably associated with the type and amount of distress of that particular road section. Although it was reported in some references that there is a relationship between asphalt hardening and cracking, no attempt was made to document such an observation on this project (*8, 9*).

Inspection of the data indicated that variations in penetration and viscosity test results on recovered asphalts were randomly located throughout the asphalt pavements studied. These observations and a consideration of the great standard deviations associated with these test data indicate that a great number of test locations would be required to discover the extent of pavements that have different test properties. In most cases, the amount of testing required would be more extensive than most agencies would consider feasible and would be of practical use only if the testing resulted in different mix designs for each section of the project that had different test properties. However, traffic loads and other factors associated with low-volume roads do not justify an extensive testing program or the breakdown of the project into separate subunits of different mix design. For practical purposes, the various layers of a particular low-volume road pavement can be scarified, ripped, or cut and mixed together, and can be considered to have an average asphalt penetration or viscosity value throughout the entire recycled section for mix design determinations.

Aggregates that were recovered from existing low-volume road pavements were found to have a similar gradation. Aggregate gradations from county roads and city streets fall within a relatively narrow gradation band (Figure 2). The statistical analyses showed some significant differences between the aggregates of the various layers that form the asphalt pavements, but a close evaluation of the results indicated that these differences were of no practical significance.

These findings mean that an average gradation (percentage of material passing the various sieves) of the layers being considered for recycling in a particular project can be used for mix design purposes. This average gradation value would serve to inform the design engineer of the required sizes and proportions of virgin aggregate that the reclaimed asphalt pavement material needs to meet a particular standard gradation for a base, subbase, or surface course mix.

The statistical analyses of the Marshall stiffness data obtained from core layers of low-volume road pavements showed that in most cases the top layer was the stiffest of the asphalt courses that formed the pavement under study. This indicated that the wearing surface was formed of stronger mixes than the lower layers, for most pavements in service.

No practical differences were found between the Marshall stiffness of county road and city street pavements, and within a particular section of road. These findings indicate that the strength and stability of the underlying layers of a candidate pavement must be carefully evaluated. If a new, recycled asphalt mix is placed on top of weak and unstable pavement layers, the structural support will be such that the distresses and faults that made the pavement a candidate for rehabilitation in the first place are likely to be evident again in a short time.

It can generally be concluded that the material characteristics of low-volume road pavements in Indiana that are candidates for recycling can be expected to have a relatively high level of variability. Some improvement can be obtained during the processing from pavement to reclaimed material through a controlled milling or reclaiming operation. For example, when recycled pavement of a better quality is needed, the project could be separated into subunits that might have different mix designs.

Finally, it is recommended that the following procedures be followed to sample asphalt concrete pavements from low-volume roads before the reclaiming process begins:

- Obtain samples and perform laboratory tests as outlined in the testing procedures section of this paper.
- Establish construction units only on the basis of aggregate gradation and percentage of asphalt, unless it can clearly be demonstrated that penetration or viscosity test properties are significantly different.
- Perform a detailed mix design study for each individual recycling project being undertaken. Final mix designs should be based on reclaimed and processed asphalt pavement material whenever possible.

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Cold Recycling of Asphalt Pavements on Low-Volume Roads

PRITHVI S. KANDHAL AND WILLIAM C. KOEHLER

Cold recycling can play a major role in the upgrade of secondary low-volume roads. Many farm-to-market roads exist in Pennsylvania that are good candidates for cold recycling with emulsified asphalt. Many roads have been built up over the years with open-graded cold mixtures (travel plant mix) or layer after layer of seal coats and surface treatments. These roads are also heavily patched. It is necessary to obtain a uniform, strengthened base course if these so-called "pancake" roads are to be upgraded and widened at the same time. Cold recycling is a good upgrading strategy to accomplish this. The Pennsylvania Department of Transportation had completed about 90 cold-mix recycling projects by the end of 1985. All three modes of cold-mix recycling have been attempted: central plant mixing, mobile plant mixing, and in-place mixing. The Pennsylvania Department of Transportation's guidelines for selecting suitable candidate projects and standard specifications for a cold recycled base course are provided. The current mix design procedures, which are at a developmental stage, are also provided. The optimum emulsion content is selected by considering (a) bulk specific gravity of the compacted specimen, (b) initial resilient modulus, and (c) resilient modulus after vacuum saturation with water. Case histories are included of some projects that were completed by use of mobile mixing plant and in-place mixing in which different equipment was used. Experience in cold recycling indicates a need for obtaining optimum moisture content in the reclaimed asphalt pavement material so that the emulsified asphalt can be dispersed effectively in the mix. The recycled mixtures are usually susceptible to damage from moisture intrusion and abrasion by traffic. It is therefore necessary to cover these mixtures with at least two applications of a surface treatment to avoid ravelling and potholing.

Pennsylvania, like many other states, is not currently building many new highways. The existing highways are being maintained and upgraded as part of the so-called 3R Program: Restoration, Resurfacing, and Rehabilitation. Cold-mix recycling can play a major role in upgrading the secondary low-volume roads. An attempt is being made to use cold-mix recycling on roads that are far from the central mix plants. No standard design procedure exists for designing these recycled mixtures, and much work needs to be done. In fact, no generally accepted, standard design procedure exists for the cold mixtures that contain 100 percent virgin aggregate. Therefore, an attempt is being made to work on the design and construction of cold recycling projects at the same time, because cold recycling appears to be an economical upgrading strategy. So far, about 90 cold-mix recycling projects have been completed.

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Cold-mix recycling is primarily being used for the base course. However, on some very low-volume roads, a 3- to 4-in depth of the existing roadway was recycled and a single seal coat was applied as the wearing course. Based on experience, at least a double seal coat must be used.

SELECTION OF PROJECTS

Many roads, like the one shown in Figure 1, are very good candidates for recycling. This road has many transverse, longitudinal, and reflected widening cracks. Many of these roads were widened in the past, and each time an overlay was applied, widening reflection cracks developed. Cold-mix recycling eliminates these cracks and provides a base with a uniform support across the entire width of the pavement. Roads have also been built up over the years with open-graded cold mixtures or layer after layer of surface treatments. These roads are also heavily patched. The base is of a very nonuniform type, and if a hot-mix overlay is simply applied on these surfaces, extensive cracking will develop. These roads are also good candidates for simultaneous recycling and widening.

Guidelines have been provided to the engineering districts that include information on the characteristics of cold recycled mixtures, the selection of projects, the estimation of quantities of materials, and construction. These guidelines aid the pavement management engineer in the development of cold recycling projects.

Standard specifications have been developed for cold recycled, bituminous base courses. These specifications are versatile and permit the use of available or innovative recycling equipment and procedures. Information on these guidelines and specifications is available from the Materials Testing Division of the Pennsylvania Department of Transportation.

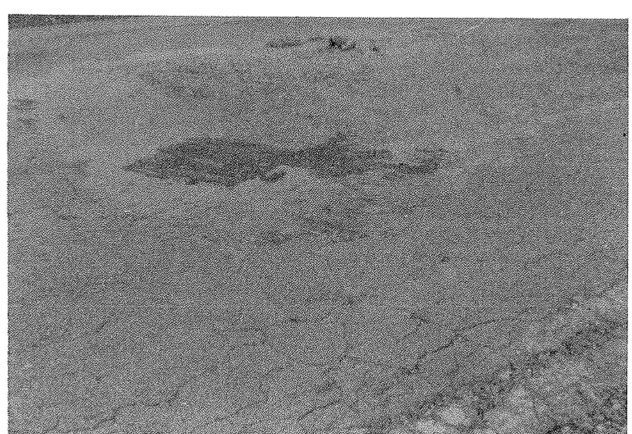

FIGURE 1 Existing road surface on the Chester County project.

MIX DESIGN

As was mentioned earlier, the mix design procedures are still at the development stage. Although it is desirable, it is not always possible to obtain a sample of the reclaimed asphalt pavement (RAP) material to design the recycled mixture in the laboratory. A RAP is defined as a removed or processed pavement that contains bitumen and aggregate. Fifteen 6-in-diameter pavement cores are usually obtained from representative sections of the project. These cores are crushed in a laboratory jaw-crusher to generate RAP for mix design purposes. Evidently, the RAP obtained during the actual recycling operation is most likely to vary in gradation and particle shape from the laboratory-made RAP depending on such factors as the equipment type and rate of milling. This requires that field adjustments be made to the design moisture content and emulsion content during construction.

The following tests are run on the RAP:

- Gradation of the RAP,
- Abson recovery of aged asphalt in the RAP to determine viscosity at 140°F and penetration at 77°F, and
- An extraction analysis to determine the percentage of asphalt content and the gradation of the aggregate in the RAP.

Based on the results of these tests, the emulsion type (CMS-2 or CSS-1h) is selected, and the need for blending a virgin aggregate is also established. If the RAP consists of a hot mix, the penetration of the recovered asphalt is usually low, such as 15 to 20. In these cases, the use of a CMS-2 emulsion that has an asphalt residue of 100 to 250 penetration is preferred. However, there are many cases in which the RAP comes from a road that was built over the years with seal coats, surface treatments, or cold mixes and that contains a relatively softer asphalt. In this case, the use of a CSS-1h emulsion that has an asphalt residue of 40 to 90 penetration is preferred. The addition of virgin aggregate to the RAP is considered if the RAP (a) consists of a sand mix, (b) contains excessive binder, or (c) does not have an acceptable gradation.

Optimum Moisture Content

Several 500-gram batches of the RAP (100 percent passing 1-in sieve) are made. The emulsion content is kept constant at 2.5 percent by weight of RAP, if 100 percent RAP is used. The initial moisture content is varied in increments of 1 percent. After the emulsion is mixed by hand for 2 min, the mix should have a ≥90 percent coating. Mixes are unacceptable if (a) they strip or stiffen excessively on mixing, (b) break prematurely, and (c) become excessively soupy and segregate on standing. A working range of optimum moisture content was established for use in the field. The RAP is maintained at ambient temperature and emulsion is used at 140° ± 10°F to prepare the mixes.

Optimum Emulsion Content

Three mixtures are each made with at least four emulsion contents using the established optimum moisture content. If 100 percent RAP is used, 2.0, 2.5, 3 0, and 3.5 percent emulsion contents are normally used. If a RAP and virgin aggregate blend is used, higher emulsion contents are attempted. The following procedures are used to prepare and test the specimens:

- The loose mixture is cured in an oven at 105°F for 45 min. It is then remixed for 30 sec and allowed to cool to room temperature.
- The specimen is compacted in a Marshall mold with 75 blows on each side. Prior to 1985, 50 blows were used. The practice was changed to 75 blows because there were some indications from the field projects that the in-place density was higher.
- The specimens are extruded from the molds on the following day, and cured in a forced-draft oven at 104°F for 3 days.
- The bulk specific gravity and the resilient modulus (M_R) of all specimens are determined at 77°F.
- The specimens are vacuum-saturated by immersing them under 1 in of water and applying a vacuum of 26-in H_g for 30 min. The vacuum is gradually released and the specimens are left submerged in water for at least 30 min.
- The specimens are removed from the water and the percentage of water absorbed during vacuum saturation is determined.
- The resilient modulus (M_R) of the specimens after this moisture conditioning is determined and the percent retained M_R is calculated.
- The Marshall stability and flow at 77°F of the moisture-conditioned specimens are determined. If the stability-flow curves do not peak, all stability values that correspond to a flow of 10 units are reported.

The optimum emulsion content is selected by considering the following test parameters:

- Bulk specific gravity of the compacted specimen,
- Initial resilient modulus (M_R),
- Resilient modulus (M_R) after vacuum saturation, and
- Percent retained M_R.

The initial resilient modulus (M_R) usually decreases as the emulsion content is increased. However, the rate of decrease of the M_R value and the M_R values after vacuum saturation are considered to establish the optimum emulsion content. No limiting values or acceptance criteria have been established for these test parameters. The optimum moisture and emulsion contents are recommended as a starting point in the field, and are subject to adjustment as needed.

CASE HISTORIES

Three modes of cold-mix recycling have been tried: central plant mixing, mobile plant mixing, and in-place mixing.

In central plant mixing, the RAP material is hauled to a central mixing plant, mixed with emulsion, and returned to the site and laid through a paver. This is usually not the most economical recycling method. A stabilization plant (Figure 2) is typically used. Segregation problems have been reported as a result of handling and transporting the RAP and recycled mix.

In mobile plant mixing, the pavement is milled and the RAP is loaded on a truck that feeds a mobile mixing plant, such as a

Midland Motopaver, where the emulsion is added and the mixture is laid.

In in-place mixing, the recycling is performed in-place by a train of equipment that mills, adds emulsion, mixes, and lays. Most cold recycling projects were performed in this manner using a wide range of equipment.

Some of the projects that were completed by means of the mobile mixing plant and in-place mixing are discussed in the following sections.

Chester County Project

The first cold recycling project was located on Gum Tree Road in Chester County near Philadelphia and a Midland Motopaver was used. This road (Figure 1) had a lot of patches and alligator cracking, which indicated a poor base. A Roto Mill was used to mill 3 in of the existing road. The RAP from the windrow was picked up to be loaded on the truck (Figure 3). The truck discharged the RAP into the Midland Motopaver where it was mixed with 2 percent CMS-2 emulsion (approximately 5 gal/ton of the RAP by weight) in the pugmill and laid through a screed (Figure 4). The recycled mix was laid to give a compacted base course of 5 in thickness (Figure 5). This compacted recycled base course was opened to traffic for a couple of weeks (Figure 6) before it was overlaid with 1-1/2 in of hot mix. This road was completed in 1980 and it carries a lot of truck traffic. So far, it is performing very well. It should be mentioned that the road was also widened during the recycling process.

Susquehanna County Projects

We completed five in-place recycling projects that totaled 16.6 mi in 1983 in Susquehanna County, which is in northeastern Pennsylvania. Most of the low-volume roads have been built up over the years with macadams, seal coats, and surface treatments and are heavily patched, as shown in Figure 7.

A Bros reclaimer, which pushes the emulsion tanker, as shown in Figure 8, was used on these five projects. The Bros reclaimer has a cutting drum and a spray bar for emulsion. It

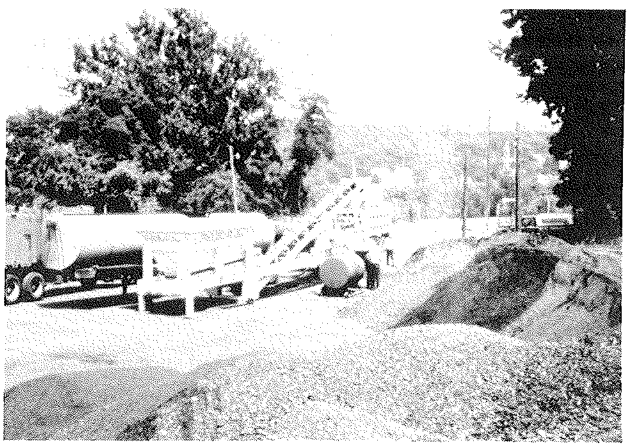

FIGURE 2 Typical stabilization plant for cold recycling.

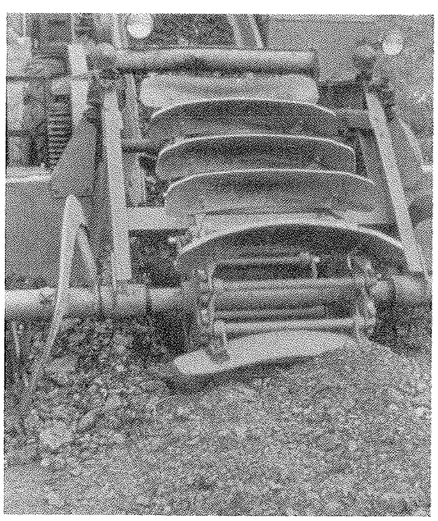

FIGURE 3 Recycled asphalt pavement being picked up from the windrow on the Chester County project.

FIGURE 4 Recycled asphalt pavement being discharged into the Midland Motopaver on the Chester County project.

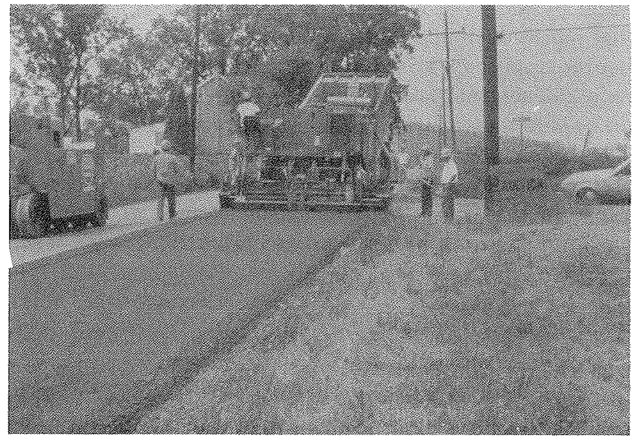

FIGURE 5 Laydown behind the Motopaver on the Chester County project.

FIGURE 6 Completed roadway on the Chester County project.

FIGURE 7 Existing road surface on the Susquehanna County project.

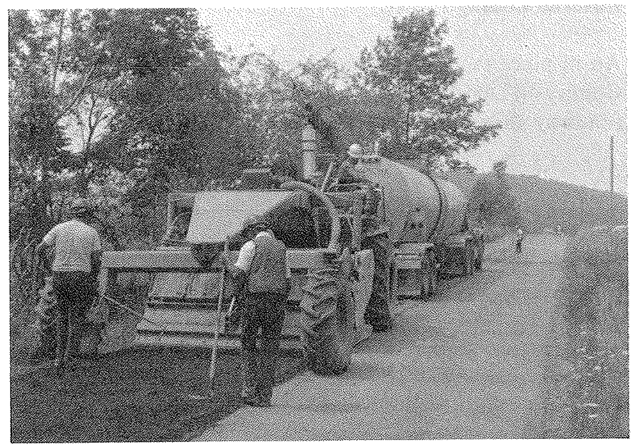

FIGURE 8 Equipment train in which Bros reclaimer is shown on the Susquehanna County project.

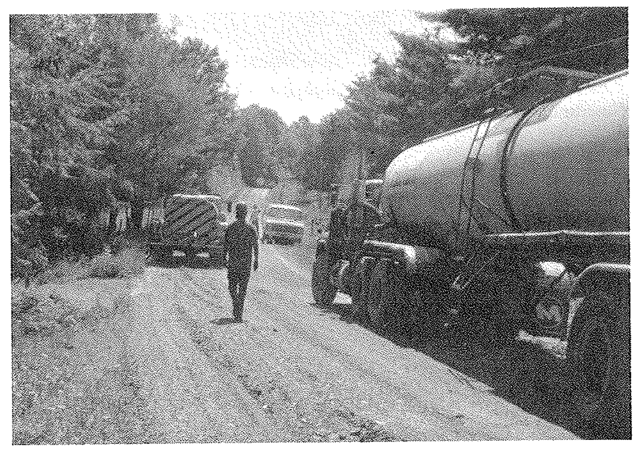

FIGURE 9 Application of water after the first pass of the reclaimer on the Susquehanna County project.

cuts and mixes the emulsion at the same time. Because about 2 to 3 percent CMS-2 emulsion was being used, it was difficult to disperse this small amount in the RAP because of the lack of sufficient moisture content that was determined in the laboratory. The water tanker shown in Figure 9 was brought in to raise the moisture content in the RAP to a range of 3 to 5 percent. The reclaimer therefore had to go over the road twice. The first pass was performed to reclaim (without adding emulsion) so that the water could be added to the loose RAP; the second pass was necessary to add and mix the emulsion.

The compacted, recycled base course appeared to be very dense. It was covered with a single application of seal coat. As was mentioned earlier, at least a double application is recommended because the cold recycled mix is generally not adequately water- and abrasion-resistant. These projects were completed in August and September of 1983. The pictures shown in Figures 10 and 11 were taken in March 1984 after a snowfall and the road surface is partially wet. The surface was good in some sections (Figure 10). However, when the single seal coat was lost, potholes like those shown in Figure 11 developed.

The inadequacy of a single seal coat is shown in Figure 12, in which the large stones of recycled base course can be seen through the seal coat. After 2 years of service, three projects were patched and a double surface treatment was applied. The condition of the road in July 1986, after this treatment was applied, is shown in Figure 13.

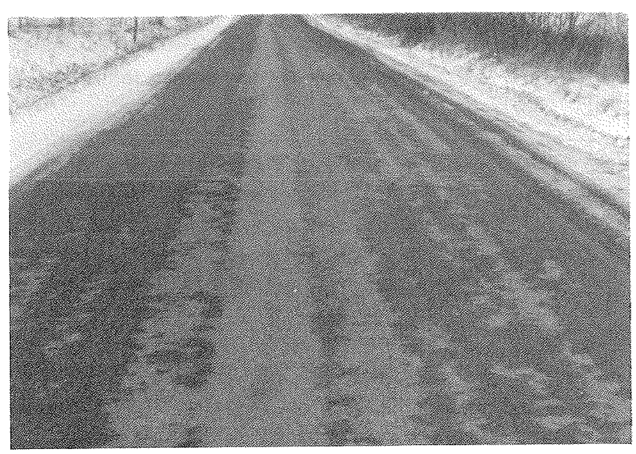

FIGURE 10 Roadway after the first winter on the Susquehanna County project (good section).

Luzerne County Project

An in-place recycling project was also completed in Luzerne County on Legislative Route 40060 in 1983. The existing 4.5-mi roadway shown in Figure 14 was narrow, badly cracked, and heavily patched, and had an unsatisfactory cross-section. A Raygo Barco Mill (Figure 15) was used for recycling. This machine has a cutting drum and two spray bars for water and emulsion. Two small water tanks with a capacity of 250 gallons each are mounted on the machine. It pushes the emulsion

FIGURE 11 Roadway with potholes after the first winter on the Susquehanna County project.

FIGURE 12 Large stones of recycled base showing through a single seal coat.

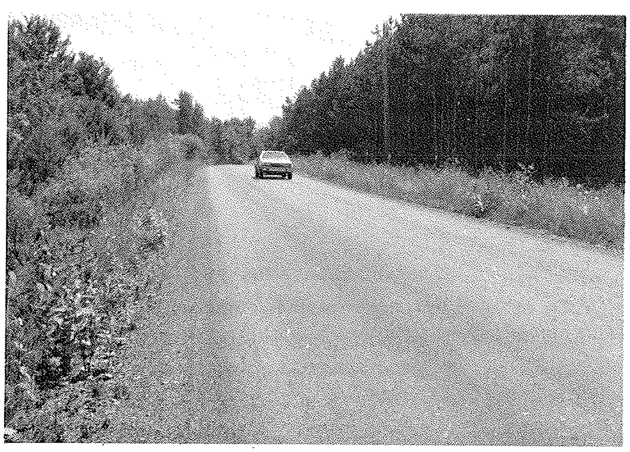

FIGURE 13 Roadway after double surface treatment on the Susquehanna County project.

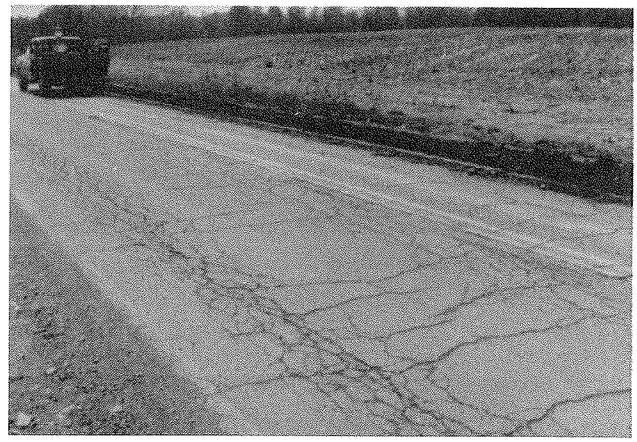

FIGURE 14 Existing road surface on the Luzerne County project.

FIGURE 15 RayGo BarcoMill equipment on the Luzerne County project.

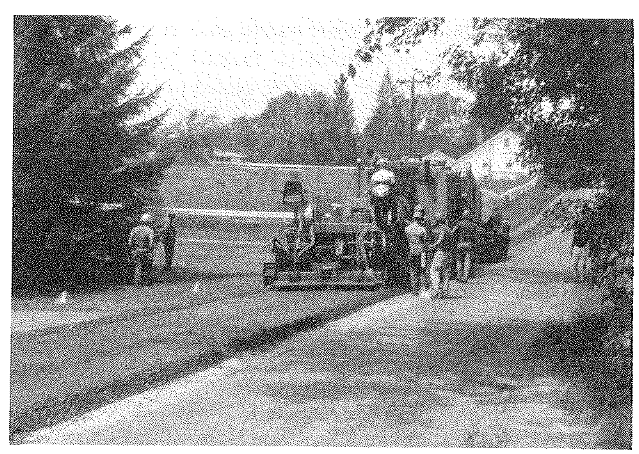

FIGURE 16 Equipment train on the Luzerne County project.

tanker in front and pulls a paver behind. The existing roadway was milled and recycled to a depth of 3 in by the equipment train shown in Figure 16. A section of the existing roadway had only a 1-1/2-in bituminous overlay. This required the addition of 1-1/2-in virgin aggregate to make up the total 3-in recycled depth. The virgin aggregate was spread ahead of the recycling train.

A CSS-1h emulsion was used on this project. Fortunately, the RAP had a substantial amount of initial moisture, because it rained before the job was started. Therefore, not much water

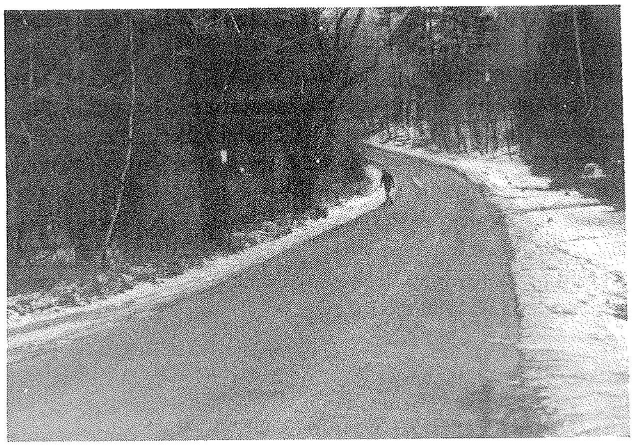

FIGURE 17 Roadway after the first winter on the Luzerne County project.

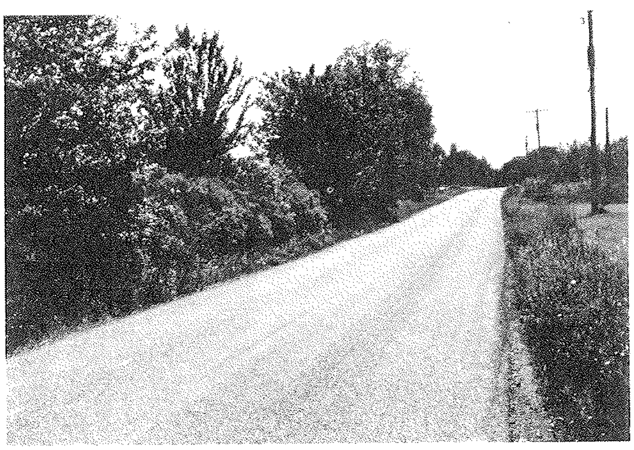

FIGURE 18 Roadway after 2 years on the Luzerne County project.

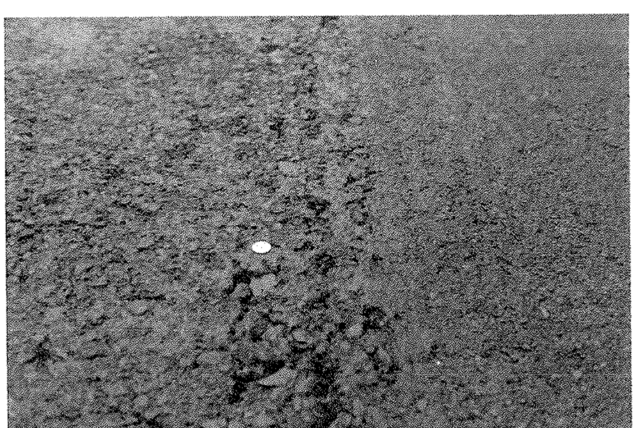

FIGURE 19 Roadway on the Luzerne County project; (l) without seal coat, (r) with seal coat.

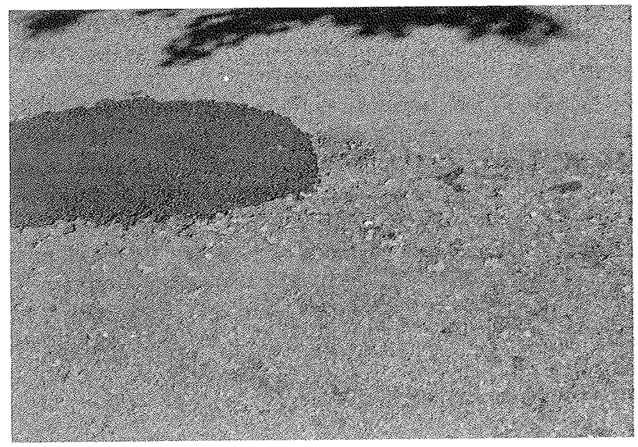

FIGURE 20 Patched pothole and adjacent section with lost seal coat on the Luzerne County project.

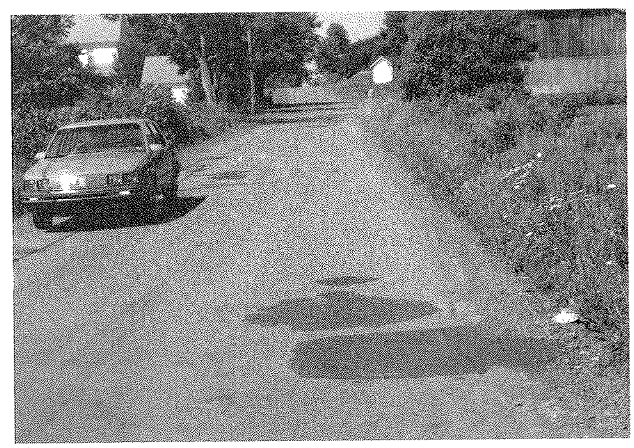

FIGURE 21 Potholed section of roadway on the Luzerne County project in which 100% recycled asphalt pavement was used.

FIGURE 22 Good section of the Luzerne County project in which a 50/50 blend of recycled asphalt pavement and virgin aggregate was used.

had to be drawn from the two small tanks, and the recycled mix was satisfactory. However, if the RAP was relatively dry, these tanks would have had to have been replenished with water very frequently, which can pose problems on these narrow hilly roads. Although this recycled project also had a single seal coat, it was performing very well after the winter of 1983 to 1984, as can be seen in Figure 17, which was taken in March 1984. The condition of the roadway in June 1985, after 2 yrs, can be seen in Figure 18. A few (5 to 6) potholes had appeared, which indicated the need for another application of a seal coat.

This project was inspected again in July 1986. A section in which the seal coat was lost can be seen in Figure 19 (left side of picture); the section is on the verge of developing a pothole. A pothole patch that was placed in such a section is shown in Figure 20.

After 3 years, the section in which 100 percent RAP was used has more potholes (Figure 21) than the section in which a 50/50 blend of RAP and virgin aggregate was used (Figure 22).

Mercer County Project

Another in-place recycling project was completed in Mercer County on Traffic Route 208 (west of I-79 London interchange) in May 1985. The recycling train consisted of an emulsion tanker, a CMI milling machine, a CMI crusher, and a Barber-Greene paver, as shown in Figures 23 through 26. The milled material passed over a 1-1/2-in scalping screen. The oversized material was fed into the crusher by a conveyor (Figure 23) to reduce its size. The material was milled and recycled to a 3-in depth. Because no provision was made for adding water before the material was mixed with the CSS-1h emulsion, the latter was diluted with water in a 50:50 ratio to provide an acceptable dispersion of the binder.

It was noted that the gradation of the RAP on this project was significantly finer than the laboratory-generated RAP. About 3 percent (approximately 7-1/2 gal/ton) CSS-1h emulsion by weight of the RAP was used. Compaction was performed with a vibratory and a pneumatic-tired roller. Because the average daily traffic on this road was 2,000 to 3,000, the recycled base course was overlaid with 3-1/2-in hot-mix overlay. The recycled pavement had been performing satisfactorily when it was inspected in 1986 after 1 year.

FIELD ADJUSTMENTS TO MIX DESIGN

As was mentioned earlier, the optimum moisture and emulsion contents from the laboratory-mix design are recommended as a

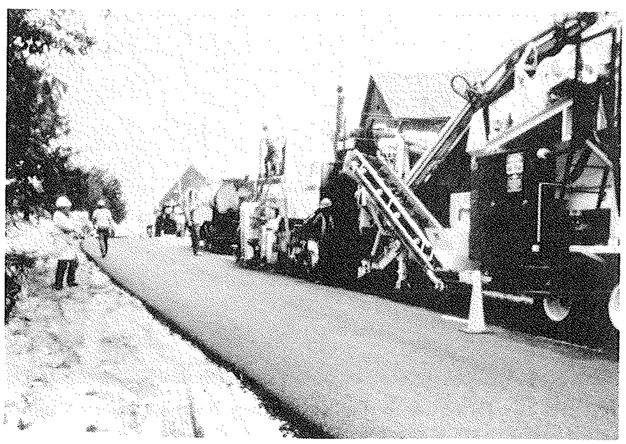

FIGURE 23 Recycling train on the Mercer County project.

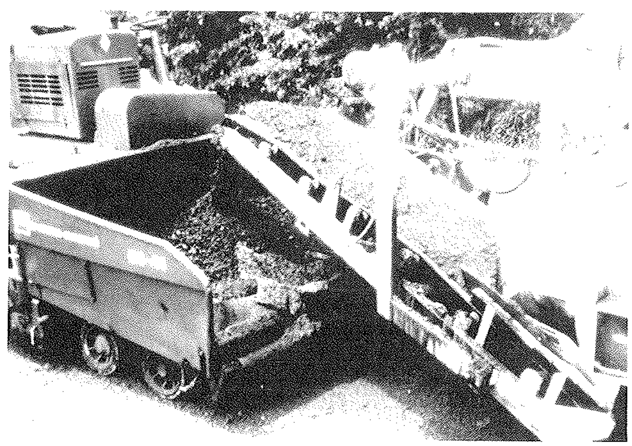

FIGURE 25 Recycled mix being discharged into the paver on the Mercer County project.

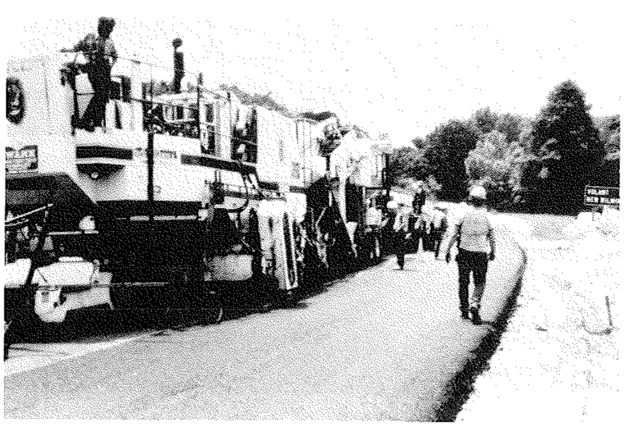

FIGURE 24 Miller and crusher on the Mercer County project.

FIGURE 26 Mix laydown on the Mercer County project.

starting point in the field, subject to necessary adjustments by persons experienced in cold recycling.

First, the coating of the recycled mix is examined after the surface dries. If the coating is not satisfactory (less than 75 percent), the moisture content is adjusted before the emulsion content. If the mix lacks cohesion in spite of an adequate coating, the emulsion content is increased. A crude test for evaluating cohesion has been used. A ball of the recycled mix is made by squeezing it in the palm of one's hand. If the ball falls apart (friable) after the pressure is released, the mix lacks cohesion. The palm of one's hand should also be examined for stains. If specks of bitumen are present, the emulsion content is generally adequate. A palm that is almost completely stained by bitumen indicates an excessive emulsion content.

The Recycling of Cold-Mix, In-Place Asphalt for Low-Volume Roads in Ohio

S. W. DUDLEY, K. MAJIDZADEH, AND K. KALOUSH

A discussion is presented of a study that was initiated in 1984 to develop specification guidelines and mix design recommendations and to obtain long-term performance data on cold-mix recycling for low-volume roads in Ohio. Future investment planning motivated the Ohio Department of Transportation to consider cold-mix recycling of low-volume roads as a maintenance alternative. Two mainline low-volume roads were selected for this study. The documentation and evaluation of the project are discussed in two parts. The first part includes the site selection criteria, preconstruction evaluation, mix designs, construction specifications, and construction monitoring. The second part includes a discussion of the performance evaluation through field inspection, data collection, and laboratory evaluation of material properties.

Maintenance of low-volume roads usually consists of patching, application of seal coats, and, in some situations, thin asphalt overlays. In most cases these maintenance procedures are only temporary and offer no corrective solution to the structural adequacy of the pavement.

In recent years, pavement recycling technology has attracted significant national attention and has become an attractive rehabilitation alternative. Cold-mix recycling in particular is an attractive option to conserve materials and energy by salvaging old pavements as stabilized bases with improved drainage, alignment, and grade.

Cold recycling can be performed either in a central plant or on-site. High production rates can be obtained from central plant recycling; mixes that contain up to 100 percent reclaimed pavement material can be produced. In-place recycling, however, is more appropriate for low-volume roads because the recycling can be done on site.

Similar equipment is used for in-place recycling as for in-place stabilization operations. In fact, the cold-mix, in-place recycling of bituminous pavement layers can be combined with the stabilization of the underlying unbound layers. The equipment that is typically used for in-place, cold-mix recycling consists of rollers, bulldozers, scarifiers, planers, milling machines, rotary mixers, motor graders, windrow devices, power brooms, self-propelled vibratory or steel-tired tandem and pneumatic-tired rollers, water distributors, and other equipment.

The only additional requirement over soil stabilization equipment is the extra power or wear resistance needed to properly size the existing bituminous pavement layers. The recent development in pulverizers, traveling hammer mills, and cold milling machines has had a significant influence on existing construction techniques and the establishment of this process as a viable option.

BINDER REQUIREMENT

Asphalt cement and emulsions have been used in cold-mix recycling since the early 1940s. Conventional equipment was used to crush old bituminous surfacings and combine the pulverized material with part of the unstabilized base or new aggregate to form reconstituted pavements.

Several binders are currently used to upgrade or stabilize existing pavements. Bituminous binders are best suited to well-graded blends of material and offer considerable benefits because of their versatility, the achievable particle bond, the

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resulting flexible strength, and the reduction in permeability. Moreover, the existing hard and brittle binders can be modified by special bituminous additives and incorporated in the production of new mixtures that perform satisfactorily. Emulsions often are the best choice because of the inherent moisture content of the treated material. Both medium-setting and slow-setting emulsion types can be used.

One of the requirements to develop a well-designed, cold-mix material is to add binder to soften the mixture and add stability and strength. The most common binders for cold-mix recycling are generally medium-curing cutbacks of the grade MC800, cement, and water. Soft or high penetration asphalts that range between 200 to 300 penetration have also been used and, in many instances, slow- or medium-setting asphalt emulsions such as CMS have been used.

MIXTURE CHARACTERISTICS

It should be emphasized that one of the justifications for the use of cold-mix recycling is the savings in energy that results from the elimination of heat from the bituminous mixing process.

The major problem to be expected in the cold-mix recycling process is mixture compaction, which stems from the lack of application of heat to the mix. It is difficult to mix the crushed aggregate, crushed pavement, binder, softening agents, and old asphalt. In order to enhance the initial mixing process, it is customary to add more water to facilitate the coating of the aggregate and asphalt binder. The poor mixing characteristics of binder and aggregates could also result in asphalt bleeding to the surface of the pavement. Cold-mixed recycling mixtures are obviously susceptible to other distress modes such as rutting, raveling, fatigue, and other failures directly associated with the strength and environmental conditions of bituminous mixtures.

MIXTURE CHARACTERIZATION

Laboratory Procedures

Mixing and coating, which are affected by construction methods as well as the mixture design, greatly influence the performance of the recycled pavements. Therefore, laboratory analyses need to be correlated with field experience to be significant. The density of the compacted mix and the percentage of absorption and the resistance to water damage are important tests. The mixture variables in cold-mix recycling are binder content, moisture content, curing time, aggregate characteristics, compaction method, and binder consistency.

No universally accepted method currently exists for curing, mix design, and sample preparation for cold-mix recycling. Specimen preparation procedures in this study followed the Purdue University and the Asphalt Institute methods, with some modifications (*1*, *2*). The following is a summary of the sample preparation and testing methods that were used:

- The reclaimed material was divided into individual samples.
- The designated amount of water was added and the materials were mixed with a spoon by hand.
- The mix was left alone for 10 to 15 min.
- The required amount of rejuvenator was added and the material was mixed both mechanically and by hand. The total mixing time was 2 min, which included a 30-sec period of hand mixing in between periods of mechanical mixing.
- The mix was cured for 1 hr in a forced-draft oven at 140°F and was then remixed for 30 sec with a mechanical mixer before it was compacted.
- The mix was then compacted into Marshall specimens using 50 blows per face.
- The samples were extruded within a reasonable period of time (24 hrs).
- The specimens were left to cure for 48 hrs before they were tested.
- Marshall testing procedures for cold mixes, as specified in AASHTO T245, were adopted.

WITCO Mix Design Procedure

The WITCO mix design method was also used to determine the asphalt demand for the recycled mixture and the type of rejuvenator to be used. According to this method, the following are the four basic properties of the existing pavement to be recycled:

- Asphalt content,
- Asphalt penetration at 25°C or viscosity at 60°C,
- Aggregate gradation, and
- Asphalt demand as determined by the following formula.

$$P = (4R + 7S + 12F)/100 \times 1.1$$

where

P = weight percentage of asphalt in the mix,
R = weight percentage of rock in the aggregate (portion retained on No. 8 sieve),
S = weight percentage of sand in the aggregate (portion passing No. 8 and retained on No. 200 sieve), and
F = weight percentage of fines in the aggregate (portion passing No. 200 sieve).

A nomograph is then used to estimate the amount of recycling agent needed to modify the viscosity of the old asphalt. This is followed by a laboratory mix design using the Marshall method to ensure that minimum requirements are met.

SITE SELECTION AND PRECONSTRUCTION INVESTIGATION

Two low-volume roads were selected for this study. The two projects are State Routes (SRs) 761 and 564, which are located in Noble County, Ohio. State Route 761 was built in 1968 as a full-depth asphalt pavement on a soil subgrade. State Route 564 was a soil-aggregate road that was upgraded with chip-seal treatments. Maps of the projects' locations are shown in Figure 1. The beginning and ending log miles of each project are shown in the following table.

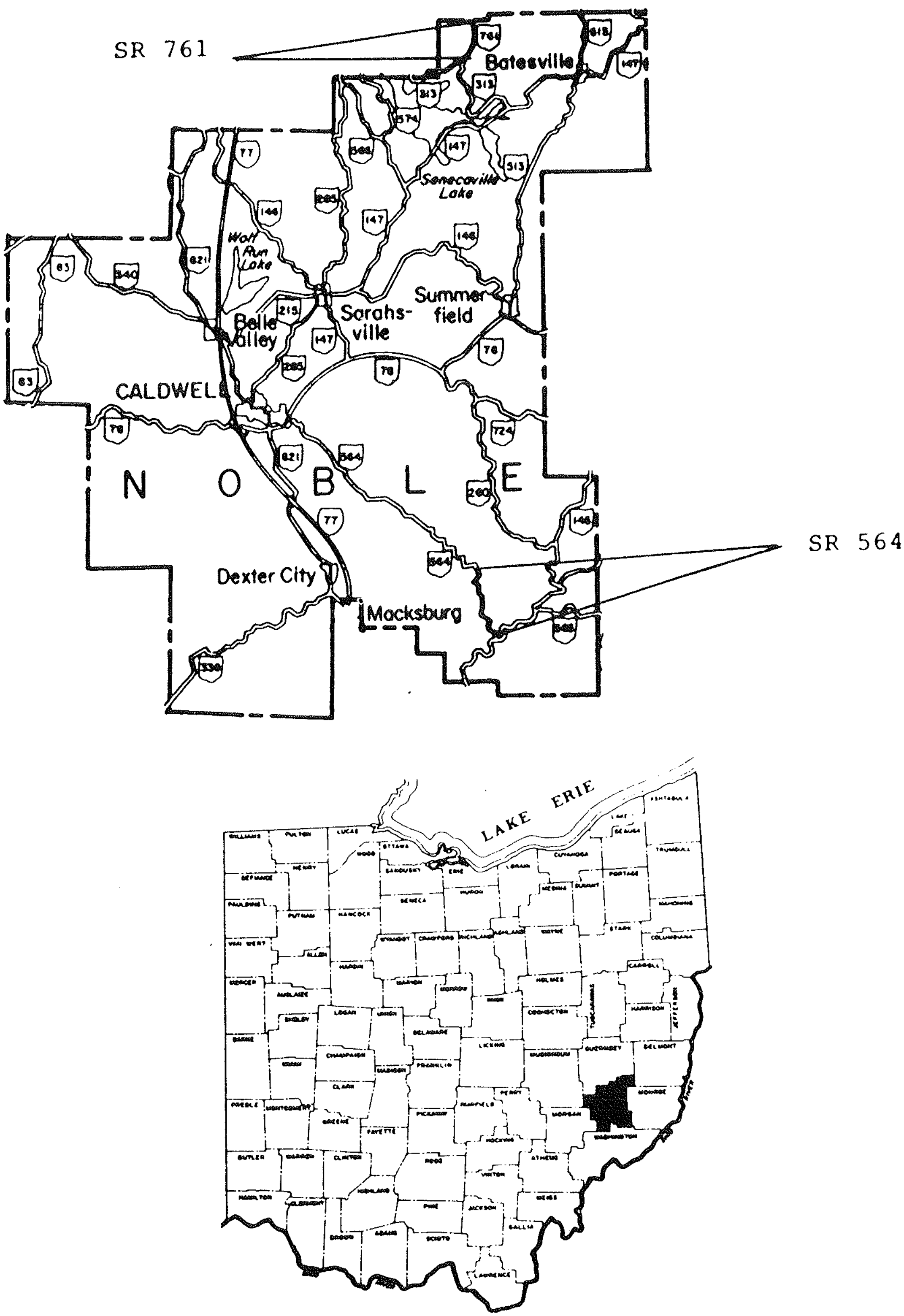

FIGURE 1 Project locations.

	Beginning and Ending Log Points	*Net Length (mi)*
SR 564	10.63-11.98	1.35
	13.05-13.70	0.65
SR 761	0.00-1.90	1.90

State Routes 564 and 761 were visited by Ohio Department of Transportation (ODOT) and Resource International, Inc. engineers on November 15, 1984. The purpose of this visit was to identify the problems in each project and verify that these projects were candidates for recycling. State Route 564 was in poor condition. Severe potholes and raveling were evident in many locations. The beginning and the ending sections of the 3-mi road were in better condition and suffered from moderate cracking and raveling. The middle 1-mi section of the road (excluded in the previous table) was reconstructed by the Department of Natural Resources in 1983. The reconstruction consisted of 6 in of aggregate base (ODOT Specification 304), 6 in of bituminous aggregate base (ODOT Specification 301), and 3 in of asphalt concrete leveling and surface courses (ODOT Specifications 403 and 404). The candidate section for recycling of SR 564 is in a high water region that had experienced occasional flooding during the last 2 years. Routine maintenance work included patching and application of tar.

State Route 761 was rated to be in poor condition. Debonding was identified in the pavement layer and severe transverse cracking was identified along the pavement length. In addition, moderate longitudinal cracks, rutting, and poor drainage were occasionally noted. It was concluded that both projects were in poor condition and were candidates for rehabilitation.

Laboratory Evaluation of Pavement Cores

Nine pavement cores with soil samples were extracted at random from SR 564 for further laboratory investigation. Three cores failed when they were removed from the core barrel. Three more fell apart when they were cut in preparation for testing. It was noted that mud layers formed between the asphalt in some cores. The thickness of the cores varied from 5 to 7 in. The asphalt content averaged 3.3 percent and the viscosity was 64 poises, according to AASHTO T202. Additional test results detected the presence of chemicals such as aliphatic oil. The Marshall stability was low and averaged 395 lbs at 77°F. These data are summarized in Table 1.

Five cores were also extracted from SR 761 for laboratory evaluation. These cores were smoothly divided into two parts that represented the debonded surface and the leveling courses. The total thickness of the pavement averaged 6 in. The asphalt content was 5.3 percent and the viscosity was 12,500 poises. These data are summarized in Table 1.

Soil classification data for both projects are presented in Table 2. The soil of SR 564 was classified as A-2-4 and the soil of SR 761 as A-6, according to AASHO classifications.

MIX DESIGN ANALYSIS AND DEVELOPMENT OF CONSTRUCTION SPECIFICATIONS

In accordance with the mix design procedures discussed previously, mix design analyses were performed on in situ materials. The aggregate gradation of SR 564 complied with specifications; therefore, no additional aggregates were necessary. Because of the low asphalt content and viscosity, it was decided to use a CSS-1H asphalt emulsion, and a cationic slow-setting emulsion with a harder base asphalt (ASTM D977-77 standard specification) as a recycling agent (*3*). Laboratory trial mixes were performed and included recompacted samples with water and mixes with the emulsified asphalt added at different percentages. Two criteria were used to optimize the mixture: Marshall stability and indirect tensile strength. The test results of these trial mixes are shown in Table 3.

Trial mixes with SS-1 emulsified asphalt, a softer asphalt base, and Cyclogen M were designed for SR 761. Analysis indicated that virgin aggregates were needed to control the total asphalt content in the recycled mix if SS-1 was to be used. The mix design analysis for Cyclogen M indicated a blending ratio of 24 percent of the rejuvenator with the aged asphalt (1.2 percent by total weight of mixture). These results are summarized in Table 4.

A thickness design analysis using the Asphalt Institute procedure for cold-mix recycling indicated that the minimum required thickness for SR 564 was 4 in with a 2-in, hot-mix surface course (*2*). This was based on a CBR value of 15 and two 18-kip equivalent axle loads per day. The minimum required thickness for SR 761 was 4.5 in with a 2-in, hot-mix surface course based on a CBR of 3 and one 18-kip equivalent axle load per day.

In summary, based on field and laboratory evaluation, it was decided to recycle the top 6 in of SR 564 with CSS-1H at 6.8 percent by total weight. The recycled base would be overlayed by a 3-in bituminous aggregate base and 1 in of an asphalt concrete surface course. It was believed that the emulsion would act as a binder and not as an agent to restore the existing asphalt properties.

It was also decided to recycle the top 4 in of SR 761 with Cyclogen M and overlay it with 1-1/2 in of an asphalt concrete leveling course, and 1-1/2 in of an asphalt concrete surface

TABLE 1 LABORATORY EVALUATION OF THE EXISTING PAVEMENT

	SR 564	SR 761	
Thickness, in	5-7	6	
AC content, percent	3.3	5.3	
Viscosity, poises	64	12,500	
Density, pcf	132	136-139	
Stability at 77°F	395	1,151-2,166	
Flow, 01 in	13	14-12	
Gradation % passing	SR 564	SR 761	Bituminous aggregate Base specification
1 in	97.4	100	75-100
3/8 in	68.3	66.2	-
No. 4	42.3	44.1	25-60
No. 8	26.0	36.3	15-45
No. 16	16.7	29.9	10-35
No. 30	10.7	20.6	
No. 50	6.4	8.1	3-18
No. 100	3.7	2.9	-
No. 200	1.9	1.7	1-7

TABLE 2 SOIL DATA

Test Results	SR 564	SR 761
Water content, percent	8.4	20.7
Unit wt., lb/ft	143.8	100.6
Percent fines, #200	3.4	67.3
Liquid limit	39	39
Plastic limit	20	27
Plasticity index	9	12
AASHO soil classification	A-2-4	A-6

course. The installation of shallow pipe underdrains was also recommended because of the poor existing drainage. Specifications and plans were developed and are available from the Ohio Department of Transportation or Resource International, Inc.

PERFORMANCE MONITORING

Construction Techniques

Construction started on September 9, 1985, and continued through that month. The existing pavement was milled and processed with a CMI Rotomill, which is a two-engine milling machine with a 12-ft-wide cutter head. The machine had the capability to mill the old surface at a variable depth up to 7 in. The material was crushed and screened through a 1-in sieve on the first trailer. Millings that did not pass the 1-in sieve were crushed again. The millings that passed the 1-in sieve were mixed with the emulsion at the second trailer. A tanker truck that preceded the CMI supplied the emulsion to a meter at which the number of gallons distributed were monitored. The mixture was then windrowed and aerated before it was spread and shaped by a grader. The material was then compacted with a vibratory roller.

The existing pavement on SR 564 was believed to be 6 in deep. During the milling process, the thickness of the pavement varied between 2 and 7 in. The depth control was easily adjusted, but in some sections the subgrade materials could not be avoided. Virgin aggregates were applied at those sections to increase the base thickness. The CSS-1H emulsion was applied at 5 gal/yd^2. In some instances, the distributor meter malfunctioned and the millings were saturated with the emulsion. This created soft sections in the road. The process was performed in two passes and because the pavement width was only 20 ft, a 2-ft section was overlapped in the second pass.

About ten 300-ft, full-depth patches were constructed with pipe underdrains on SR 761. The subgrade was so soft that the construction equipment could not proceed. Other sections in the road deteriorated badly. During the milling operations, the remaining 2 in of pavement did not support the CMI as well as expected. Cyclogen ME (emulsified Cyclogen M) was distributed and mixed with the pavement material at 1.5 gal/yd^2. In comparison with SR 564, the mixture was noticeably softer. The recycling agent was sprayed over the compacted pavement as an additional seal.

Traffic was allowed to return to the recycled pavement for at least 7 and up to 12 days, during which time some occasional rain was experienced. The surface was then swept and given a tack coat. It was noticed that some soft areas had failed and there were wheel track depressions in both pavements. Four inches of hot mix were applied on SR 564 and 3 in on SR 761.

Material Properties of SR 564

Fifty-two cores were extracted along the length of the project on November 11, 1985. All extracted cores were 4 in. in diameter and extended through both the overlay and the recycled layer. Some difficulty was encountered when drilling through the recycled layer at sections that were earlier identified as having soft pavement material. However, representative cores from those sections were extracted successfully. The cores were cut and measured for thickness and unit weight. Measurements indicated that the overlay thickness was an average of 4.2 in and the recycled layer thickness ranged from 1.7 to 6.0 in with an average value of 3.5 in. The asphalt content averaged 10.2

TABLE 3 TEST RESULTS OF MIX DESIGN TRIALS FOR SR 564

Percent CCS-1H	Unit, wt. pcf	Stability 140° F	Stability 77° F	Tensile Strength, PSI
0	142.9	-	-	13
4.3	137.3	-	-	74
5.2	134.0	528	3524	-
6.8	135.5	427	4841	61
8.3	134	454	3015	85

TABLE 4 TEST RESULTS OF MIX DESIGN TRIALS FOR SR 761

AC	Percent Virgin Aggregates	Unit Wt. pcf	Stability at 140/77° F	Tensile Strength
Percent SS-1				
5.8	40	123.4	50/2025	21
6.3	30	121.9	114/1995	25
7.8	0	125.6	108/2226	54
Percent Cyclogen M				
1.2	0	133.1	210/3444	32

percent with a standard deviation of 3.2 percent. The average gradation is shown in Figure 2. The CSS1-H emulsion samples that were collected during construction had a Saybol viscosity of 35 S (77° F) and met the specification requirements (20 to 100 S).

A laboratory testing program was undertaken to evaluate the recycled pavement cores. Tests were conducted and the results are shown in Table 5. In addition, 40 samples were fabricated using the same materials and proportions that were used in the field mix. The laboratory samples were subjected to the same testing program and the results are also shown in Table 5.

The coefficients of variation indicate that there is scattering in some of the test data for both field and laboratory samples. Variations in gradation and asphalt content from sample to sample is typical for cold mixes. In addition, exact weight by sample splitting is not feasible because milled materials are tested and aggregates are not. Therefore, differences in weight and height from sample to sample are inevitable.

The modulus of resilience and indirect tensile strength values for both laboratory and field samples of SR 564 were generally good. Lottman's accelerated moisture damage test was used to determine the durability of the recycled pavement to freezing and thawing. The resilient modulus and the tensile strength for field cores were reduced by 41 and 46 percent, respectively, when subjected to freeze and thaw cycles. These values can be compared to 59 and 42 percent, respectively, for specimens fabricated in the laboratory. Although no minimum value criteria were agreed on for fracture toughness and stability, the results of these tests are considered satisfactory for base courses.

An immersion-tension test was performed to simulate the effect of water (140° F) on the tensile strength of the compacted mixtures. Test results were satisfactory and showed a 24 and 35 percent reduction in strength for field and laboratory samples, respectively.

Material Properties of SR 761

Two attempts were made to extract cores from this project on November 14 and December 17, 1985. Neither attempt was successful in obtaining intact cores. The cores crumbled when they were removed from the core barrel. It was noted that only the top 0.5 to 1 in of the recycled layer was intact. The remaining 3 in failed. It was suspected that the excess moisture in the rejuvenator did not have enough time to escape and adequate stability values were not reached before the overlay was applied. Cyclogen ME samples that were collected during construction were tested for Saybol viscosity at 77° F. The average viscosity value was 11.7 S and did not meet the minimum requirement of 15S. This could have been another indication that excess moisture was mixed with the rejuvenator.

Forty laboratory samples were also fabricated with the same materials and proportions that were used in the field mix for SR

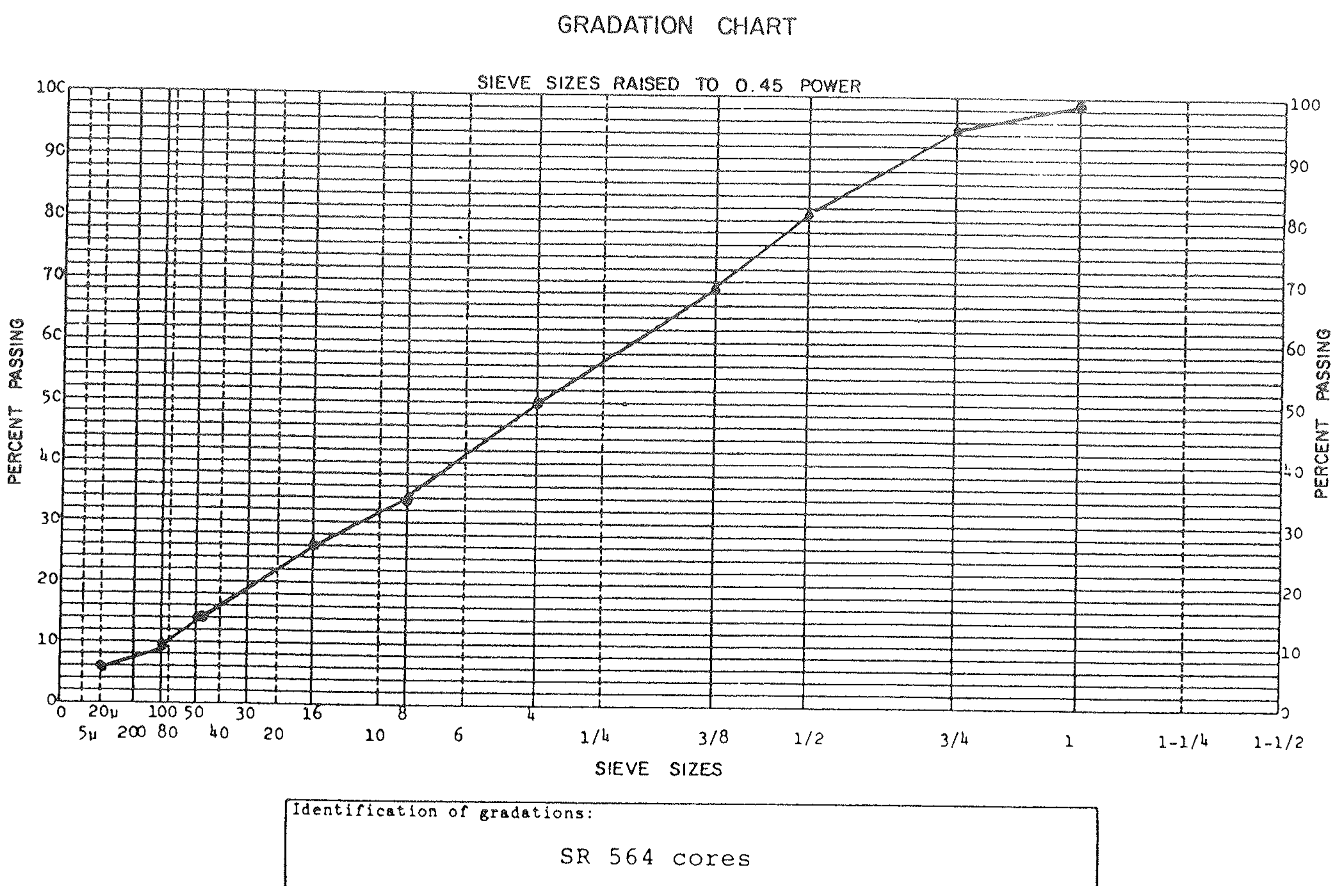

FIGURE 2 Percentage passing various sieve sizes for SR 564.

TABLE 5 PHYSICAL AND ENGINEERING PROPERTIES OF LABORATORY AND FIELD CORES

	LABORATORY SAMPLES SR 564				FIELD CORES SR 564				LABORATORY SAMPLES SR 761			
TEST	n	$\bar{x}$	σ	C.V%	n	$\bar{x}$	σ	C.V%	n	$\bar{x}$	σ	C.V%
Thickness, in	40	2.62	0.05	2	N26	2.49	0.07	3	40	2.51	0.04	2
					S26	2.28	0.34	15				
Unit weight, pcf	40	133.6	0.90	1	N26	134.4	3.9	3	40	132.9	1.15	1
					S26	130.8	3.8	3				
MR @ 70°F (x10^6psi)	11	0.66	0.18	27	28	0.88	0.48	55	18	0.29	0.15	52
Durability Tests**												
MR @ 70°F (x16^6psi)	6	0.44	0.18	28	6	1.02	0.3	29	6	0.38	0.21	55
MR @ 70°F (x16^6psi), (a)	3	0.50	0.17	34	3	0.67	0.08	11	3	0.25	0.11	44
MR @ 70°F (x16^6psi), (b)	1*	0.18	-	-	3	0.6	0.35	58	3	0.17	0.03	18
σ_y @ 70°F, psi	3	50.3	1.7	3	4	67.5	12.4	18	3	34.3	1.25	4
σ_y @ 70°F, psi (a)	3	47.7	3.2	7	3	57.9	6.9	12	3	17.4	5.1	29
σ_y @ 70°F, psi (b)	1*	29.1	-	-	3	36.6	13.7	37	3	11.1	2.1	19
KIC (psi in)	4	292	45.8	16	4	191	71.3	37	3	158.3	2.62	2
Immersion Tension												
σ_y before, psi	3	50.3	1.7	3	4	67.5	12.4	18	3	34.3	1.25	4
σ_y after, psi	3	32.6	8.6	26	3	51	1.5	3	1*	13.15	-	-
Stability, @140°F, lbs	3	408	36.8	9	4	493	94.3	19	3	227	22.4	10
Stability, @ 70°F, lbs									3	3360	540	16

* Two samples failed
** Lottman's procedure
(a) Vacuum saturation only
(b) Freeze and thaw

n = No. of samples
$\bar{x}$ = Average value
σ = Standard deviation
C = Coefficient of variation

761. Ten samples were prepared to study the effect of curing on the mixture. Samples were placed in an air draft oven at 140°F for 1, 2, and 3 weeks. Curing was an important factor to consider for SR 761 samples.

The test results for SR 761 laboratory samples showed a 55 percent reduction in modulus of resilience, a 68 percent reduction in tensile strength when subjected to a freeze and thaw cycle, and a 62 percent reduction in immersion-tensile strength test. Stability values at 77°F were acceptable, but values at 140°F were not. All test results were generally 50 percent lower than those of SR 564 samples. Specimens cured at 140°F showed a significant increase of 93 percent in modulus of resilience and an increase of 82 percent in tensile strength after 2 weeks. Those percentages increased to 138 and 124, respectively, after 3 weeks, as shown in Figures 3 and 4.

A second set of field cores from both projects will be collected 1 year after construction, and will be subjected to a similar testing program.

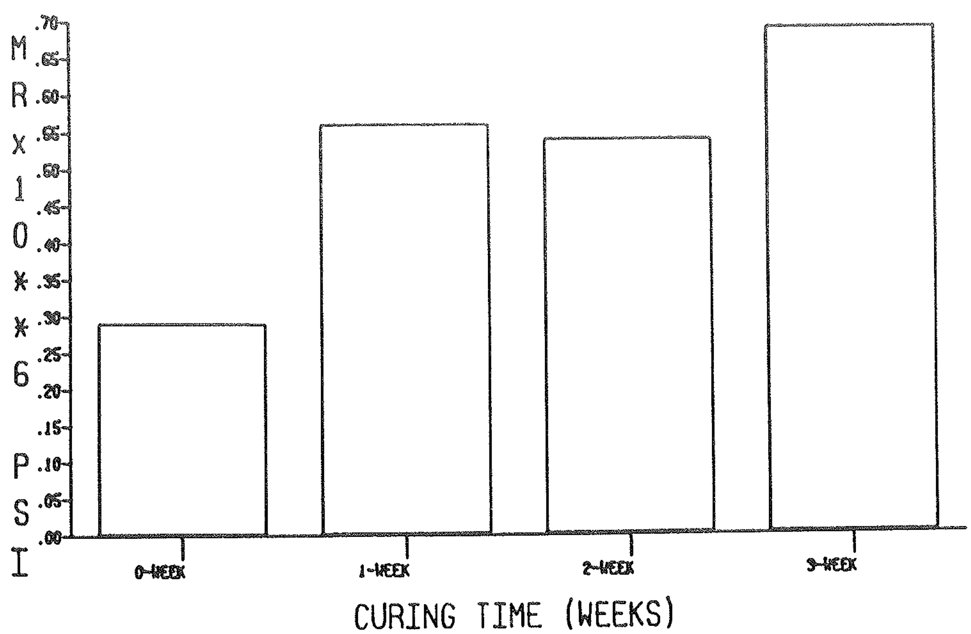

FIGURE 3 Modulus of resilience versus curing time for SR 761.

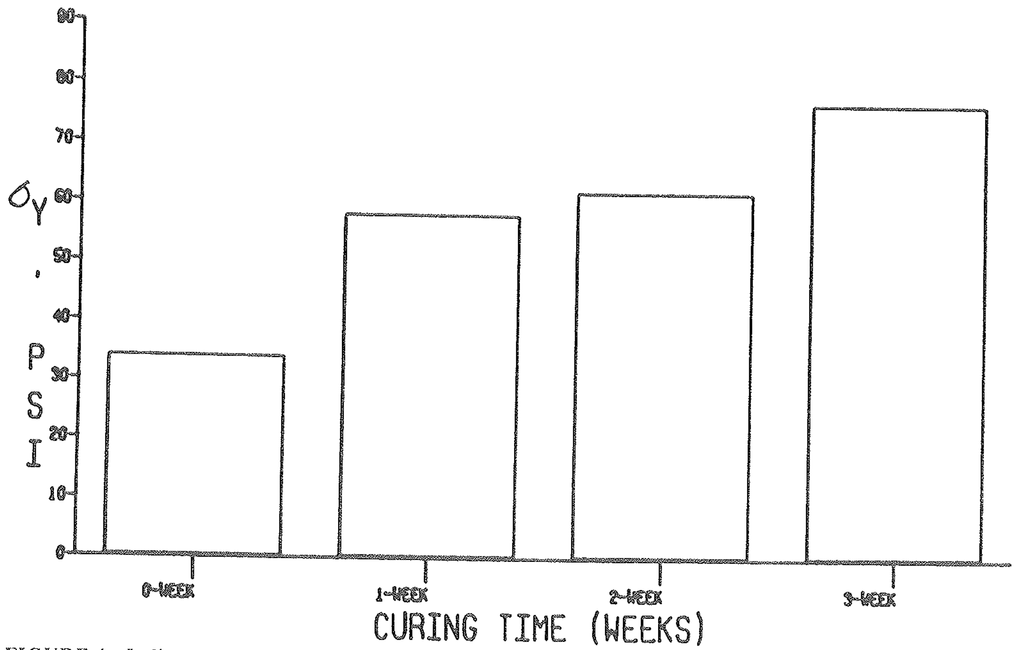

FIGURE 4 Indirect tensile strength versus curing time for SR 761.

Condition Survey

A surface condition survey of SR 564 before the overlay was applied revealed that the recycled layer had a tendency to ravel at the curves. Two soft sections were identified for wheel track densification. The emulsion saturated the mixture during construction on the first section; the subgrade material mixed with the emulsion because of a lack of pavement thickness on the second section. Other sections were in good condition. State Route 564 was visited 1 and 6 months after the overlay was applied. The pavement performed adequately and there was no indication of any failure in the recycled base underneath it. A visual inspection of SR 761, before the overlay was applied, revealed rutted sections under the wheel path and occasional raveling. Two severely damaged sections were compacted, broomed, and sealed before the overlay was applied.

State Route 761 was visited 1 month after the overlay was applied. Problems were noted in only one location. A 20-ft section that had failed earlier had rutted and was severely cracked in the wheel paths. The cracks had been sealed by district maintenance personnel. The project showed no other problems 6 months after construction.

Deflection Measurements

A Dynaflect deflectometer was used to measure deflections on the two pavements on December 5, 1985. The air temperature on that day was 40° F, and the pavement temperature was 52° F. These data are summarized in Table 6.

The maximum deflection (W1) is an indication of total pavement structure and its support conditions (*4*). Weak support conditions are generally associated with an increase in the maximum deflection. A range of 0.7 to 1.0 milli-inches is considered to represent the transition from satisfactory to unsatisfactory performance. The recycled section of SR 564 indicated an average maximum deflection value of 0.56 milli-inches, which is a comparable value for the adjacent newly constructed section. Both sections were considered satisfactory, as shown in Figure 5. State Route 761, however, had an average maximum deflection value of 1.09 milli-inches, which indicated an unsatisfactory performance. A plot for W1 versus stations is shown in Figure 6. It should be noted that the maximum deflection was reduced at locations at which full-depth asphalt patches were constructed for drainage.

The pavement's spreadability is a function of the pavement's

TABLE 6 DYNAFLECT DEFLECTION MEASUREMENTS

	SR 564				SR 761				Newly constructed** section SR 564			
	W1	S%	SCI	BCI	W1	S%	SCI	BCI	W1	S%	SCI	BCI
Average	0.56	62.7	0.14	0.04	1.09	58.2	0.28	0.13	0.56	67.1	0.12	0.06
Low	0.37	51.9	0.09	0.02	0.45	44.4	0.08	0.05	0.35	61.8	0.08	0.03
High	0.91	70.0	0.22	0.08	1.98	74.3	0.63	0.26	0.73	71.5	0.15	0.08
SD*	0.15	4.2	0.03	0.02	0.41	7.4	0.14	0.05	0.10	2.8	0.02	0.01

* SD Standard Deviations

** Section reconstructed in 1983 6" Aggregate base
6" Bituminous aggregate base
3" Asphalt concrete

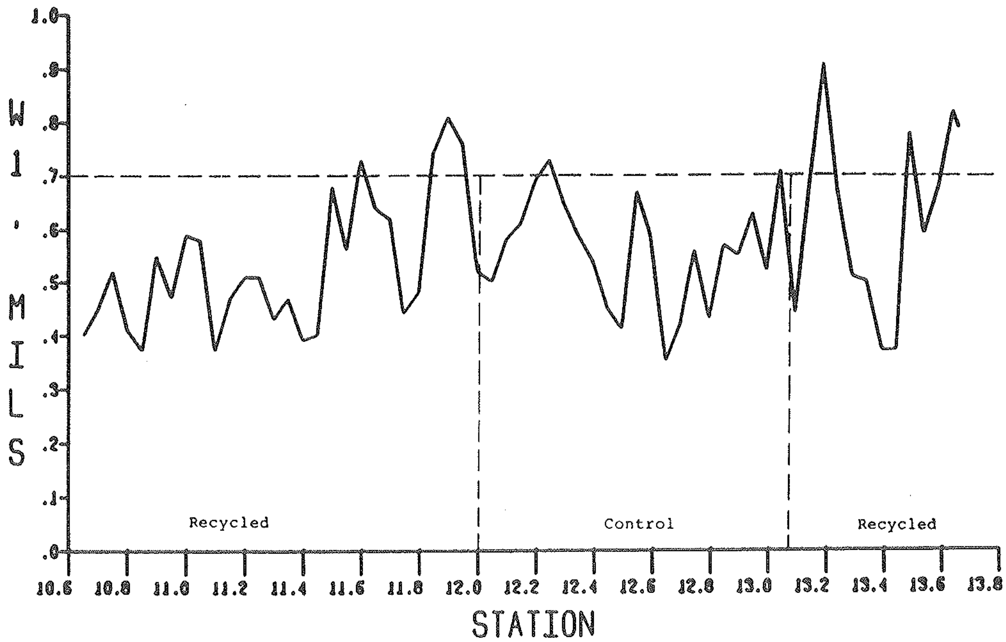

FIGURE 5 Maximum deflection versus station for SR 564.

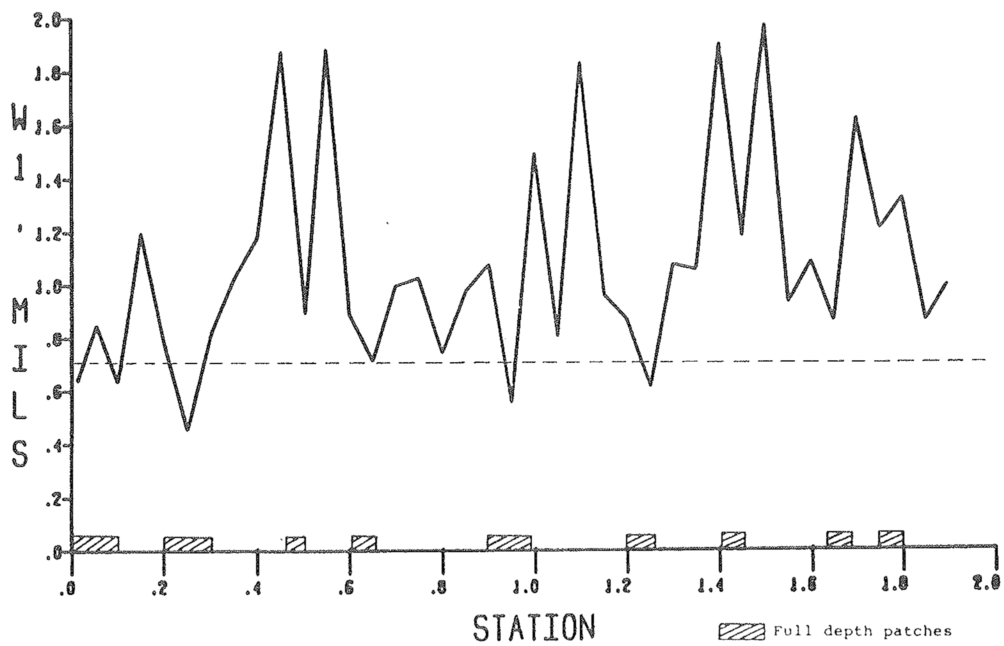

FIGURE 6 Maximum deflection versus station for SR 761.

stiffness and its ability to distribute loads. Typical values for spreadability usually range from 60 to 75 percent. The spreadability values for SR 564 and SR 761 are considered satisfactory and marginal, respectively.

The Surface Curvature Index (SCI) is inversely proportionate to the radius of curvature and directly proportionate to the tensile stresses and strains at the bottom of the asphalt layer. In flexible pavements, SCI values greater than 0.25 milli-inches are considered unsatisfactory and indicate poor pavement conditions. The SCI values of SR 564 were satisfactory, but the SCI values of SR 761 indicate poor performance.

The Base Curvature Index (BCI) is a parameter used in deflection of subgrade problems and base support conditions. Values greater than 0.15 correspond to poor subgrade and base conditions. The BCI values of SR 761 indicated poor conditions, but the BCI values of SR 564 indicate satisfactory conditions.

It can be concluded that the structural performance of SR 564 is considered satisfactory and better than that of SR 761. Deflection data will be collected periodically to detect any change in the performance of both routes.

INITIAL CONCLUSIONS

Based on the initial laboratory test results, field condition evaluation, and deflection measurements of both projects, the following initial conclusions can be made:

- The performance of SR 564 (recycled with CSS-1H) is satisfactory.
- The performance of SR 761 (recycled with Cyclogen ME) is unsatisfactory. However, this should not be taken as an indication that the concept of cold recycling or the use of chemical rejuvenators are responsible for the road's poor performance. The lack of an adequate soil structure and curing time proved to be major reasons for the unsatisfactory performance of this project.
- A second set of field cores and deflection measurements should be collected 1 year from the date of construction to detect any changes in the performance of the recycled layer.
- The general appraisal of everyone involved in these projects was favorable to the concept of recycling low-volume roads into stabilized bases. Construction problems and future pavement performance will enable the Ohio Department of Transportation to modify and improve construction specifications for future projects.

ACKNOWLEDGMENT

This research was sponsored by the Ohio Department of Transportation. The authors would like to thank the Bureau of Construction, Research and Development, and Maintenance for its cooperation in obtaining the cores and deflection measurements.

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Cold, In-Place Asphalt Pavement Recycling

DEAN A. STEWARD

Many rural, low-volume asphalt roads constructed since 1955 have a rough riding surface, show signs of thermal cracking, and are in need of rehabilitation. Laboratory tests were initiated to discover which oil-based products, in addition to those currently marketed, would produce workability and Marshall test values in a recycled mix equivalent to those commonly used in conventional cold-mix construction. Three additives were selected for use in an experimental project. The additives varied from a marketed cold-mix rejuvenator to a gas/oil with a high paraffin content. A recycle train was used to gain precise control of additive content and the reclaimed asphalt pavement (RAP) gradation, and because a high production rate would lessen the inconvenience to the traveling public. Test data from the project, such as ambient and mix temperatures and the RAP gradation, were compared to laboratory test data. The pavement was tested to determine density and asphalt properties during and at the completion of the project. Testing is being repeated annually to document any changes in those values. The actual pavement condition is also being visually inspected to determine which additive is most effective in retarding thermal cracking in the recycled pavement.

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The pavement to be recycled was a 24-yr-old, 4-in thick, cold, road-mixed, blade-laid bituminous base in which a sand-gravel aggregate and cutback asphalt were used as binders. Transverse thermal cracks allowed water to enter, which resulted in depressions adjacent to the cracks. Maintenance resealing had added about 1 in of thickness to the pavement. Severely depressed sections adjacent to the transverse cracks had also received a 3-in overlay of cold mix similar to that used in the original construction. The life of the thin overlays had been only 3 to 5 yrs, by which time the pavement surface again became rough.

Because of the limited amount of funds available for the maintenance and construction of low-volume paved county roads, a search began for a low-cost additive that could be mixed with the pulverized reclaimed asphalt pavement (RAP). This would enable the mixture to be relaid and form a stable, relatively waterproof pavement without the cracks that were allowing water to enter the subgrade.

The rip, crush, and relay method of recycling, in which 4 to 6 percent water is added to aid compaction, had been practiced for several years with pavements less than 3 in thick. The results were acceptable, except that the lack of waterproofing had caused some failures and random cracking was observed after 1 year.

The prices that were quoted for the milling, pulverizing, and mixing of an additive indicated that cold recycling with a recycle train would probably be more cost-effective than an overlay. Oil-based additives were selected for testing to determine which additive would best waterproof the recycled pavement and soften the aged asphalt.

SAMPLING AND TESTING OF IN-PLACE PAVEMENT

In 1983, in-place pavement depth, density, asphalt content, penetration, and aggregate gradation were determined from cores taken at 1/4-mi intervals from the center of alternate traffic lanes. The results are as follows:

- The pavement was 4 in deep,
- The asphalt content was 5.4 percent (ASTM D2172),
- The asphalt penetration was 29 (ASTM D5), and
- The pavement density was 135 lbs/ft^3 (ASTM D1188).

The aggregate gradations after the asphalt was removed, as specified in ASTM D2172, follow. The sieve sizes, first in inches and then in numbers, are followed by the percent retained on the sieve screen in parentheses: 1(0), 3/4(1), 1/2(2), 3/8(4), 4(11), 8(23), 16(40), 30(60), 50(77), 100(86), and 200(89).

Several additives were mixed with RAP to determine the amounts and types of additives that would visually produce the workability necessary for a road mix, motor grader laydown type of construction. In 1984, a laboratory was employed to run tests using a modified Marshall procedure to compare the density, stability, and flow of the RAP mixed with various amounts of several additives. The additives are described in a later section.

The pavement was pulverized by a laboratory machine that had shown a pulverization comparable to that obtained in the field with a high production milling and profiling machine of the same make as that used on this project. More detail on this is provided in a later section.

Procedure 1, a modified Marshall test procedure in which room temperature was used for mixing and 110° F was used for compaction, was used in an attempt to approximate expected field conditions with a paver for laydown. The steps that must be followed in Procedure 1 are as follows:

- Crush pavement chunks.
- Weigh crushed RAP material into 4,500-gram samples.
- Blend RAP and additive for 1-1/2 min in mechanical mixer.
- Heat RAP to 110° F.
- Mold Marshall specimen with 50 blows at 110° F using equipment specified in ASTM D1559.
- Extrude Marshall specimen from mold after 30 min using equipment specified in ASTM D1559.
- Cure specimen for 24 hrs at room temperature.
- Obtain density as specified in ASTM D1188 (not wax-coated).
- Place specimen in 140° F blower oven for 2 hrs (oven specified in ASTM C88).
- Run Marshall stability test as specified in ASTM D1559.

A comparison of the reclaimed asphalt pavement gradations in the laboratory and in the field varied as follows: 1 to 6 percent and 3 to 14 percent respectively for a 3/4-in sieve, and 57 to 62 percent and 57 to 70 percent respectively for a No. 8 sieve. No variation was found in the laboratory or in the field for a 1-1/2-in sieve. The field average for a 3/4-in sieve was 6 percent; the field average for a No. 8 sieve was 67 percent. The higher retained amounts in the field were consistently recorded during the coolest temperatures.

The second procedure to approximate motor grader laydown was the same as Procedure 1, except the RAP and additive were left in an oven at 110° F for 16 hrs before the Marshall specimen was molded. The RAP and additive were remixed thoroughly by hand before the specimen was molded.

Procedure 2 produced uniformly lower stability values for most of the oil-based additives. The lowered stability was judged to be of little significance in predicting pavement durability. The use of either procedure to compare the relative stabilities and densities of various mixes resulted in the same conclusions, except when the additive had kerosene in it. In that case, the removal of volatiles resulted in a higher stability when Procedure 2 was followed.

The following design parameters were chosen: a visual workability sufficient for motor grader or paver laydown; a modified Marshall stability of 200 or more; a density of 127 lbs/ft^3 (pcf) or more; an asphalt penetration that was raised from the existing 29 into the 60 to 150 range; and a recycled pavement that exhibited as much resistance to water as possible with a minimum cost. The design parameters chosen had produced good results in new pavement construction in which cutback asphalt and similar aggregate were used in road mix construction from 1966 to 1985.

Additives

The following additives were tested with the RAP: SAE 10W40 motor oil; used motor oil; SAE twenty-weight lubricating oil stock; gas/oil; ARA-1 emulsion with 1 to 3 percent water added, and without water added; and RA-100 with 20, 25, 30, and 35 percent kerosene. The RAP with 2 to 5 percent water and RAP with nothing added were also tested.

Test data indicated that the unused SAE 10W40 motor oil was acceptable. However, its price was \$4/gal; a similar, but lower-priced oil product was therefore sought.

A commercially available SAE twenty-weight oil produced test data that were similar to both new and used motor oils. However, its price was \$2/gal, so the search continued for a lower-priced, oil-based product.

A used motor oil product that consisted of other oils and some solvents was found, but the moisture and solids had been removed by heating and settlement. The price was \$.75/gal. This product had acceptable test results but was eliminated because of possible environmental contamination, the difficulty of testing the contamination, and the fact that the U.S. Environmental Protection Agency was in the process of determining whether or not to classify it as a hazardous waste.

A gas/oil by-product available from a local refinery had a visual appearance similar to used motor oil and a price of \$.75/gal. Other properties are listed in the following table:

Viscosity at 140° F (cSt)	13.3
Pour point	65° F
Flash point (ASTM D92)	370° F
Chemical Analyses (%)	
Asphaltenes	1.0
Polar compounds	3.4
First acidaffins	9.1
Second acidaffins	24.5
Paraffins (saturates)	62.0

This product produced satisfactory test data similar to that of the other oil products.

ARA-1 is a marketed, oil-in-water emulsion that is an additive in cold recycling. Its specifications are summarized in Table 1.

RA-100 is a marketed, oil-based, hot recycling additive. Its specifications are summarized in Table 2. A 500 to 520°F end-point kerosene was blended into the RA-100 at the refinery at 25 percent by weight.

The density and stability parameters were met by the RAP with all additives tested; the highest density and stability values were obtained with water at a 5 percent rate. Modified Marshall test data are summarized in Table 3.

These results verified earlier field results of a water-bound, recycled pavement that exhibited excellent stability when dry but failed during extended periods of wet weather because the optimum moisture content was exceeded.

A 125°F versus 110°F compaction temperature comparison

TABLE 1 ARA-1 EMULSION SPECIFICATIONS

Property	Method	Requirements
Viscosity at 25°C (SSF)	ASTM D88	15 mininum, 100 maximum
Miscibility	ASTM D244	No coagulation or separation
Sieve test (%)	ASTM D244	.010 maximum
Residue[a] (%)	ASTM D244	60 minimum
Particle charge	ASTM D244	Negative
Tests on Residue From Evaporation		
Asphaltenes (%)	ASTM D4124	1.0 maximum
Saturates (%)	ASTM D4124	30 maximum
Flash point	ASTM D92	375 minimum
Thin-Film Oven Test (TFOT) weight change, (%)	ASTM D1754	4 maximum
Viscosity at 60°C. (cSt)	ASTM D2170	75 minimum, 250 maximum

[a]The residue was determined by the evaporation method described in ASTM D244, except that the sample was maintained at 300°F until foaming ceased, and then was cooled and weighed.

TABLE 2 RA-100 SPECIFICATIONS

Property	Method	Requirements
Viscosity at 140°F (cSt)	AASHTO T201	5,000-15,000
Flash point, COC, °F	AASHTO T48	450 minimum
Tests on residue from TFOT at 325°F		
Viscosity ratio[a]		3 maximum
Loss of heating (%)	AASHTO T179	2 maximum
Chemical		
Maltenes ratio[b]	ASTM D2006-70	0.2-1.2
Saturates weight (%)	ASTM D2006-70	30 maximum

[a]Viscosity ratio equals TFOT viscosity at 140°F ASTM; original viscosity at 140°F ASTM.
[b]Maltenes ratio equals $PC + A_1/S + A_2$; where PC = polar compounds, A_1 = first acidaffins, A_2 = second acidaffins, and S = saturates.

TABLE 3 A COMPARISON OF THE LABORATORY-MODIFIED TEST DATA OF VARIOUS ADDITIVES

	Percentage of Additive	Stability Procedure 1 (lbs)	Average Density (pcf)
ARA-1	1.5	300	128
Gasoline/oil	.90	290	132
Gasoline/oil	.75	348	130
RA-100 with 25 percent kerosene	1.5	220	127
Used oil	.75	330	131
10W40 oil	.75	284	127
Water	5.0	648	133
Nothing added		407	120

Note: The density was determined by ASTM D1188, the penetration by ASTM D5, and the recovery of asphalt by ASTM D1856.

test was performed to determine the relative effect that the mix temperature at the time of compaction had on density and stability. The 125° F mix compaction temperature resulted in a 3 to 5 percent increase in density and a 35 to 45 percent increase in stability (see Table 4). The three additives that were selected for use in the experimental project were ARA-1 at 1.5 percent, RA-100 with 25 percent kerosene at 1.5 percent, and gas/oil at 1 percent. These three additives were selected to compare the long-term differences in transverse cracking of the recycled pavement using additives with very different chemical compositions and costs.

The ARA-1 emulsion is marketed as a cold recycling rejuvenator and was the most expensive additive. The RA-100 additive is a marketed, hot-mix rejuvenator cut with kerosene, and was the next most expensive additive. The gas/oil additive had a high paraffin content and a low price, and was chosen to demonstrate whether its chemical would have a marked difference in the prevention of future cracking.

It was hoped that the experimental project would determine the importance the chemical analysis of an additive has in preventing transverse cracking in recycled pavement. It was also hoped that the project would determine which specifications were feasible and necessary to produce a durable and cost-effective recycled pavement.

DESCRIPTION OF PROJECT

In July 1985, the Thomas County Highway Department began to recycle a 6-mi segment of Thomas County Highway 439 (Gem Road) beginning 7 mi east of Colby, Kansas, at U.S. Highway 24, then north 6 miles. The road segment was broken down into 6 miles, referred to as Mile 1 at the south to Mile 6 at the north end for ease of reference. The width of the existing pavement top was 23 ft with 2-ft-wide earth shoulders. The pavement was constructed in 1961 as a 3-in-thick road mix base composed of 5-1/4 percent cutback asphalt and a local sand-gravel aggregate.

Maintenance seals accounted for about 1 in of additional thickness, which brought the total average pavement thickness to 4 in. Miles 1, 2, and 3 had received an additional thin overlay that raised the total thickness to 4-1/2 to 5 in. The pavement surface that had not been overlaid was rough as a result of depressions at the transverse cracks at 15- to 30-ft intervals. No rutting or shoving was evident and only eight small pavement repairs could be seen. Average daily traffic varied from 320 vehicles in Miles 1 and 2 to 220 vehicles in Mile 6. A large portion of farm and oil field trucks used all six miles. Construction trucks used the southbound (west) lane in Miles 1 through 4, hauling from a sand pit located at the 3-1/2 mi point.

EQUIPMENT SELECTION

The types of equipment used included a recycle train (Figures 1, 2, and 3) that consisted of a large milling machine with a 12-1/2-ft wide mandrel that pulled two semitrailers. The first trailer had a 1-1/2 in vibratory screen and a crusher that reduced the oversized material to a size of less than 1-1/2 in. The second trailer had a weigh-in-motion belt scale that was electronically connected to a variable-speed asphalt pump that proportioned the additive into the RAP at the front of a large horizontal pugmill. The pugmill then deposited the mixed material in a windrow on the subgrade.

Additional equipment included motor graders, a 12-ton pneumatic roller, a 7-ton double-drum vibratory roller, asphalt tank trucks that transported the additive, an asphalt distributor, a chip spreader, dump trucks, and a loader for miscellaneous work.

CONSTRUCTION SEQUENCE

In the first step of construction, the recycle train processed the pavement. After the recycle train processed both lanes, a motor grader bladed the two windrows into one, and mixed and dried it. A motor grader and rollers then laid the pavement down in 1-mi segments. Finally, a seal coat was applied after all 6 mi had been laid.

The recycle train made a 12-1/2-ft-wide cut up one side of the road and a 10-1/2-ft cut on the other side. The depth of the cut was 4 in or to the subgrade where the pavement depth was less than 4 in. The sizing and milling quality was excellent and the additive rate was closely controlled. The train production rate was one lane wide and 1-3/4 to 2 mi per day.

The two windrows of mixed material deposited by the train were combined with a motor grader into one large windrow. The windrow was aerated and mixed a minimum of 1 mi per day to remove moisture and make the mixture more uniform.

The mixture was spread and shaped by a motor grader that spread the material in lifts an average of 1/2 in or less in depth. The moisture content of the mix at the time it was laid down was .50 percent or less. The grader was followed by a pneumatic roller. This operation progressed at the rate of 1 mi every 2 days. A faster rate was possible by laying thicker lifts; however, this resulted in an undesirable rough surface texture.

The initial compaction was performed by a 12-ton pneumatic roller that made successive passes that overlapped at least half of the roller width. After the 4-in-thick material was laid down and compacted, the vibratory roller began to make successive passes that overlapped at least half the roller width. The double-drum vibratory rolling took place in the late afternoon im-

TABLE 4 THE EFFECTS OF THE COMPACTION TEMPERATURE ON THE STABILITY AND DENSITY OF THE MODIFIED MARSHALL TEST SPECIMEN

Additive	Additive Amount (%)	110° F Compaction Stability (lbs)	Density (pcf)	125° F Compaction Stability (lbs)	Density (pcf)
Gasoline/oil	.50	279	123.8	391	129.8
Gasoline/oil	.75	257	126.5	348	130.0
Gasoline/oil	1.00	200	128.4	277	132.0
Gasoline/oil	1.25	166	130.0	241	134.2

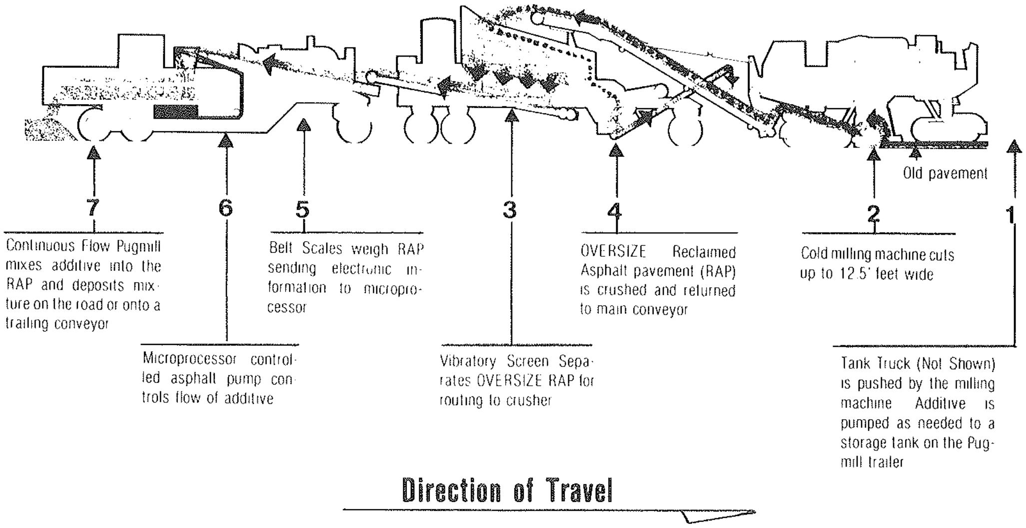

FIGURE 1 **Recycle train.**

FIGURE 2 **Screening and crushing unit.**

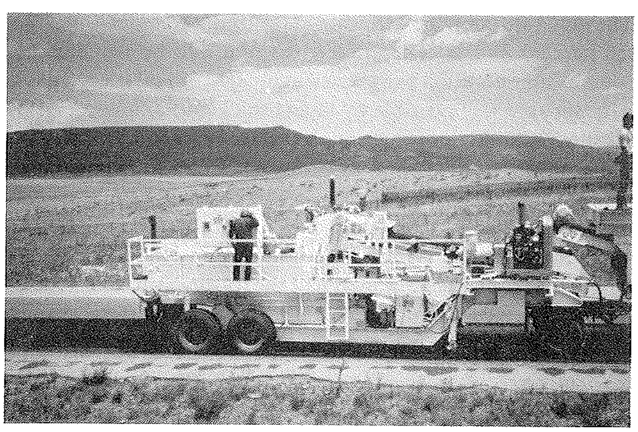

FIGURE 3 **Pugmill unit.**

mediately after the material was laid down. A second double-drum vibratory rolling took place approximately 24 hrs later. The temperature of the mix after it was laid down on a clear 90°F day was in the 100 to 110°F range. However, the mix temperature on the second day of vibratory rolling, with the same weather conditions, was in the 115 to 125°F range at a pavement depth of 2 to 3 in. Laboratory testing, field testing, and a visual inspection on the second day verified that increased density and stability were achieved by this procedure. A density comparison of the vibratory rolling sequence was conducted with a nuclear density meter when the temperature of the mix was at 115°F. The density reached a maximum after four passes, then simply leveled off with successive passes.

Previous experience with cold mixes indicated that a seal was necessary. A sand seal was therefore applied that consisted of cutback asphalt at the rate of 1/4 gal/yd^2 and a local sand aggregate.

COSTS

The total project cost was $193,000.00, or $32,167.00/mi ($2.28/yd^2). The section in which ARA-1 was used cost $34,700.00/mi; the section in which RA-100 with 25 percent kerosene was used cost $32,600.00/mi; and the section in which gas/oil was used cost $25,900.00/mi. For the sake of comparison, a project that was contracted in 1984 to place 1-1/2 in of hot mix overlay on an adjacent state highway cost $35,000.00/mi. In 1985, county forces placed 3/4 in of cold mix overlay, including a seal, at a cost of $17,500.00/mi. Two similar 1986 county recycling projects, which included paver laydown, cost $23,500.00/mi, including a seal. See Table 5 for more information.

OBSERVATIONS

Because the history of cold, in-place recycling is relatively short, a detailed consideration of different methods, materials, machinery design, and quality control of the construction process is required. A discussion is provided in the following sections of some of the factors that are important to the quality of the recycling pavement.

Testing

The comparison tests that were performed before, during, and after construction have yielded information from which future

TABLE 5 COMPARISON OF ECONOMIC DATA OF VARIOUS ADDITIVES

Additive	Road Sections Used	Price per Gallon ($)	Percent of Additive Used	Additive Cost per Mile ($)	Construction Cost per Mile ($)
ARA-1	Miles 1 and 2	1.36	1.50	13,300.00	34,700.00
Gasoline/oil	Mile 3	0.75	0.90	4,500.00	25,900.00
RA-100 with 25 percent kerosene	Miles 4, 5, and 6	1.15	1.50	11,200.00	32,600.00

specifications can be written to help define limits for contractors bidding on this type of project besides ensuring a quality, finished product for the owner. Sufficient preconstruction testing and correct interpretation of data are necessary to be able to predict with accuracy the successful construction of a cold, in-place recycling project. Of prime importance is the need to obtain a sufficient number of tests to ensure representative results. Not only is the uniformity of the recycling operation important, but also the existing pavement condition. For instance, patched spots with a high asphalt content may cause failures in the recycled pavement as a result of instability.

A test for mix workability or consistency is desirable. It has been observed on other recycling projects that hot-mix pavements and pavements with high asphalt contents cause the mix to be stiff and can cause an uneven laydown because of the lack of sufficient workability. Cold-mix pavements have had better workability than hot-mix pavements; this is believed to be caused by the diluent that remained in the cold-mix pavement.

Milling, Sizing, and Mixing

It is important to accurately control the depth of cut and proportioning of the additive. This was easily accomplished by the recycle train. Control of the gradation of the milled material is believed to be important in obtaining good density and preventing segregation. Control by specification is recommended. The only gradation problems that occurred during construction were the result of a temporary spillover caused by a partially plugged screen or equipment breakdown. The pugmill or mixer must have a sufficient capacity to ensure a uniform mixture of the additive and RAP. Care should be taken to avoid milling into the subgrade, which contaminates the RAP with undesirable subgrade material. Sections of the windrow that are excessively contaminated should be replaced with suitable material.

Blending, Additional Mixing, Drying, and Combining Windrows

Additional mixing is needed only to evaporate excess water or diluent, or if the mixture of additive and RAP is not uniform. If the processing was done with sufficient accuracy, the asphalt content of the existing pavement is uniform, and no excess moisture is present, it appears possible to eliminate any additional mixing and use a paver with a pick-up attachment to lay down the mix. The results of two projects in 1986 in which a paver was used to lay down the material have confirmed this. If the recycled mix is to be laid with a paver, the addition of water to an emulsified additive or to cool the milling machine teeth should be kept to a minimum to prevent an excess of moisture in the recycled pavement.

Spreading and Shaping

Blade or motor grader laying is an operation that depends on the ability of the motor grader operator. Good operators produce a riding surface equal in quality to a paver-laid pavement. Conversely, an unskilled operator will produce a predictably rough, uneven, and unsatisfactory riding surface. A paver-laid mix is an attractive alternative that eliminates many of the problems caused by rain. However, the mixture must be uniform and workable enough to compact the lift thickness laid, and must have a moisture content of no more than 1 percent greater than the existing pavement, with no visible free moisture.

Compaction

Mix workability, mix temperature, compactive effort, and lift thickness are primary factors in the construction of a strong, durable pavement. Laboratory mixing, compaction, and testing by the modified Marshall method accurately predicted the densities obtained on this project and could be used to set a density specification. Laboratory design testing procedures that accurately predict a field outcome should be used. Density can be predicted with reasonable accuracy by using a rolling sequence determined by a nuclear density meter and by closely monitoring mix temperatures. As determined by cores taken 1 month after laydown, all in-place densities exceeded 94 percent, and in most cases exceeded 96 percent, of the modified Marshall density obtained from field samples just before the pavement was laid down. The mix temperature at time of compaction greatly affects the density and stability obtained and is one of the primary factors to be considered in the recycled mix design, and construction specifications and methods.

Weather

Testing and experience have shown that the best weather conditions for cold mixing are hot and dry. Over 10 inches of rain fell during this project and temperatures were cool. This caused work delays, the additional drying of windrows, and patching of soft subgrade areas with cold mix. Water under the windrows required that they be moved and aerated several times. The recycle train began on July 22, 1985, and completed processing in 7 working days. However, because of the record rainfall, it was August 21, 1985, before laying was completed on Mile 1. All 6 mi of pavement were laid down by August 31. The temperatures between August 21 and 31, 1985, were as follows:

	Overnight Lows (°F)	*Daytime Highs (°F)*
Coolest	47	67
Warmest	64	100
Average	57	89

Daytime highs of 80°F were necessary to achieve good workability for the gas/oil mix, and daytime highs of 90°F were necessary for the ARA-1 and RA-100 with 25 percent kerosene mixes. Good drainage is necessary to prevent subgrade failures. Cuts through windrows at ponding areas are recommended if rain is forecast.

The most coarse gradations of the milled material occurred during the cool mornings. It was noted that the processed material from the pugmill had a temperature 15° to 25°F greater than the pavement ahead of the milling machine. The recycle train should be operated only when the atmospheric temperature is above 50°F and weather conditions are not foggy or rainy to avoid excess moisture and to ensure that the desired gradation is obtained.

SUMMARY

The use of the cold, in-place recycling has been found to be both economical and feasible. The theoretical softening of the asphalt by different additives as determined by tests on the extracted asphalt is of limited value. These values can only be related to the actual performance of the recycled pavement to be of value in predicting the life of the pavement. The condition of the pavement, particularly transverse cracking, is being observed annually, and any differences between the sections with different additives are being noted. In addition, cores are being taken annually to check the density and compare extracted asphalt properties. See Table 6 for comparative data.

ACKNOWLEDGMENTS

The author would like to thank the many people who were involved in the successful completion of this experimental project. Special thanks go to the following people, who unselfishly shared their expertise, time, ideas, and support. Clarkson H. Oglesby, Professor Emeritus of Civil Engineering, served as an inspiration to undertake this effort. Rodney Maag, P.E., Asphalt Research, Kansas Department of Transportation, was of great help in answering questions and furnishing information. Robert Morrissey, P.E., Federal Highway Administration, contributed much encouragement and support in the search for low-cost additives. J. McDaniels, P.E., and Al Hubbard, S.E.T., Allied Laboratories, helped and encouraged the study effort by presenting ideas as the testing progressed. Bill Eberly of Brown and Brown Contractors provided much help and information.

Special thanks also go to Chris Bieker of the Thomas County Highway Department, who served as the crew leader and finish motor grader operator on this project, for the many hours he worked on the project and the quality results that were obtained.

TABLE 6 COMPARISON OF THE EFFECT OF DIFFERENT ADDITIVES ON DENSITY, PENETRATION, AND DUCTILITY

	Penetration					Ductility		Density				
Additive	Additive (%)	Original Pavement	Aug. 85	Nov. 85	Nov. 86	Original Pavement	Nov. 86	Original Pavement	Marshall Design	Sept. 85	Nov. 86	Remarks
ARA-1	1.5	29	125	93	61	5	17	135	128	128.6	135.7	No cracks, January 15, 1987.
Gasoline/oil	.90	29	153	136	86	5	15	135	132	132.3	140.1	No cracks, January 15, 1987.
RA-100 with 25 percent kerosene	1.5	29	68	54	46	5	6	135	127	127.9	133.9	First transverse cracks appeared in February 1986.

The Roads Maintenance Management Program of Zila, Bangladesh

PAUL E. CONRAD

The overall goals of the Zila roads project are to increase agricultural production, particularly food grains, and to improve rural access by institutionalizing an effective program to maintain and construct low-volume rural roads annually. The work program is being performed by Wilbur Smith and Associates in association with Bangladesh Consultants Ltd. and the Public Administration Service. The scope of this ongoing program encompasses technical assistance in tendering the development of improved standard specifications for road construction, the development of a road construction manual, the oversight of design and construction activities, and an extensive training program that includes formalized training in Bangladesh, on-the-job training, and overseas training in both Bangkok and the United States. One of the main purposes of the project was to formulate road maintenance programs for the Rangpur, Sylhet, and Faridpur districts of Zila. Detailed road inventories were conducted, and performance standards were devised for 22 basic activities. Workloads, in terms of crew-days, equipment, and material resources, were formulated for 13 maintenance areas within the three districts. Resource needs for the maintenance programs were quantified, recommendations were developed for staffing and equipment assignments, and the 13 programs were costed and budgeted for the fiscal year July 1, 1985, to June 30, 1986. A detailed methodology was developed to program, schedule, and execute the work program, including a reporting system and an evaluation of results.

One of the main goals of the Zila Roads Maintenance Improvement Project is to design and install a system to maintain local roads. Regular, periodic maintenance operations are currently not being performed at the local level. So-called maintenance activities relate primarily to the rehabilitation and reconstruction of existing roads. The following steps were followed in the design and implementation of a maintenance management system in each of the three districts of Rangpur, Sylhet, and Faridpur:

- The network of roads to be maintained was defined;
- Information was collected on separate road sections;
- The roads were classified;
- The type of maintenance work required on each road classification was identified;
- A performance standard was determined for each activity;
- The maintenance level to be achieved was established;
- The number of crew-days needed for each activity on each road in an annual cycle was determined;
- Resource requirements, including nonroutine costs, were calculated;
- Unit prices were established and resource requirements were budgeted;
- The budget was approved;
- The crew-day calculations were converted into a work program;
- The work was scheduled according to the seasons and priorities;
- The work was assigned and resources were allocated;
- The work was executed, inspected, and controlled;
- Reports were returned in the same terms as the performance standards; and
- The reports were analyzed and the next program was adjusted.

NETWORK DEFINITION

A list of the roads in each of the three districts in Zila was made in early 1983 to define and initiate the maintenance program for the fiscal year that started July 1, 1985, and ended June 30, 1986. Individual roads were grouped by geographic area within each district into subdistricts, or maintenance areas.

Physical inventories of the roads commenced in July 1983. Although the inventories were hampered by weather, shortage of vehicles, and other problems, they were mostly complete by the end of the wet season in 1984. The inventories provided data on road lengths, surface type and width, shoulder width, surface condition, and major and minor drainage structures. A copy of the inventory form that was used is shown in Figure 1. A total of 137 active roads were inventoried, the total length of which was 1,094 mi. The number and total lengths of active roads in each district are listed in the following table.

District	*Active Roads*	*Total Length (mi)*
Rangpur	60	641.36
Sylhet	39	175.03
Faridpur	38	277.95
Total	137	1,094.35

The field surveys indicated that a number of road sections were interrupted and sometimes even isolated by unbridged water crossings and other gaps, which made the continuous recording of lengths impossible. This and other factors caused some long routes to be divided into sections with distinct numbers. All gaps within each numbered road were measured to record the uninterrupted length of the road.

A route numbering system was established so that the terminal place names would not have to be called out whenever a road was referred to. A sample road list in which the numbering system is shown can be found in Figure 2.

Wilbur Smith and Associates, 1301 Gervais St., P.O. Box 92, Columbia, S.C. 29202.

ODOMETER READING		
BEGIN	END	TOTAL

Road Name: ________ ____ ________

LOG MILE			SURFACE				SHOULDERS				EMBANKMENT				MAJOR STRUCTURES				MINOR STRUCTURES				SIDE DRAINAGE	TOTAL RATING	REMARKS
Odometer Reading	Actual Mileage	Sect. Length	Type	Cam-ber	Width	Cond (30)	Type	S - S Width	Cross Fall	Cond (25)	Fill Ht.	Side Slope	* F.L.	Cond (10)	Type	Width	Length	Cond (15)	Type	Size	Length	Cond (10)	Cond (10)	(100)	

SKETCH

EQUIPMENT:

Vehicle with Odometer

Locke Level or Abney Level

Cloth Tape

Magnetic Compass

Clip Board

Level Rod

CATEGORY

Preventive Maintenance ☐

Corrective Maintenance ☐

Road Improvement ☐

Reconstruction ☐

FIGURE 1 Road inventory card.

NILPHAMARI MAINTENANCE AREA

TERMINI	ROAD NO.	SECTION	LENGTH KM.	LENGTH MILES	SURFACE TYPE	CODE
Gakulpur - Chilahati	RNI-4	A	3.15	1.96	EA	45 A
		B	.45	0.28	RP	21
		C	5.15	3.20	EA	45 A
		D	4.40	2.73	EA	45 A
		E	4.50	R&H System		
		F	0.20	0.12	HBB	25
		G	0.60	0.37	RP	21
		H	0.30	0.19	HBB	25
		I	0.25	0.16	RP	21
		J	0.05	0.03	HBB	25
		K	0.55	0.34	RP	21
		L	1.55	0.96	HBB	25
		M	6.65	4.13	EA	45 A
		N	26.70	16.59	EA	45 A
		O	3.40	2.11	EA	45 C
Kishoreganj-Abuliarhat	RNI-11	A	12.05	7.49	EA	45 D
		B	5.90	3.67	EA	45 D
		C	2.80	1.75	EA	45 D
Kishoreganj-Darwani Textile Mill	RNI-12	A	0.05	0.03	HBB	25
		B	1.10	0.68	RP	21
		C	6.80	4.22	EA	45 A
		D	8.75	5.44	EA	45 A
Nilphamari-Ramganjhat	RNI-12X	A	0.05	0.03	WBM	41
		B	7.85	4.88	EA	45 A
WDB Road-Khaimarihat	RNI-19	A	1.45	0.90	HBB	25
		B	6.40	3.98	EA	45 B
Kishoreganj-Bakshiganj	RNI-30	A	6.75	4.19	EA	45 A
		B	2.50	1.55	HBB	25
		C	0.65	R&H System		
		D	5.75	3.57	EA	45 D
		E	1.50	0.93	HBB	25
		F	4.15	2.58	EA	45 A
Kishoreganj-Barobhita	RNI-32	A	5.65	3.51	EA	45 A
		B	0.55	0.34	HBB	25

FIGURE 2 Basic road list—Rangpur district.

The road inventory provided the basic data needed to develop the first maintenance program. Because changes were being made on some routes under other aspects of the work program, such as rehabilitation, widening, and new bridge construction, the new characteristics were used in all cases in which improvements were funded and would be completed by May 1985.

ROAD CLASSIFICATION

Many factors affect the types and amounts of maintenance work needed on a road segment. Paved surfaces require different activities than unpaved surfaces; old pavements need more maintenance than newer pavements. Therefore, the roads had to be classified by factors that affected maintenance activity.

The climate in Bangladesh is sufficiently uniform that this factor would not generate differences in types of work by climatic region. The small variations in topography also can be ignored, with one exception. Longitudinal ditch maintenance was required in the rolling terrain of Sylhet that was not required in the other two districts, in which roads are almost always on embankments. Little variation was consequently anticipated in classes of maintenance as a result of differences in climate and topography. Major characteristics that had to be taken into account were the type and width of surfacing on a section of road, and the age and condition of that surfacing. Seven distinct surface types were identified, including the following:

- Rigid pavement (portland cement concrete),
- Flexible pavement (bituminous),
- Herringbone bond (brick on edge),
- Single-layer pavement (cobble or brick),
- Waterbound macadam,
- Gravel or other selected material, and
- Earth.

All of the rigid pavement on the Zila roads (25.37 mi) was over 10-years old, and most of it was in poor condition and deteriorating. A single category was established for rigid pavement and designated surface type (ST) 21.

The bituminous paved roads (175.01 mi) ranged in condition from new to destroyed. Several subclassifications were considered that were based on age or construction method, but they were discarded as impracticable because insufficient data existed to establish the date of construction or last rehabilitation, and because several combinations of base and surface types existed. Although traffic volumes, and especially truck loadings, are important factors in projecting maintenance, little information was on record for the roads in Zila. Therefore, it was decided to establish two broad categories of flexible pavement, good condition and poor condition, for which different patching rates and other work could be forecast. These categories were designated ST 22A and ST 22B, respectively.

The herringbone brick (HBB) pavements formed a total length of 89.76 mi, most of it in the Rangpur district (65.50 mi). Surface Type 25 was assigned to HBB. Little single-layer pavement existed in the three districts. The cobble surface, or boulder soling, of two road sections in Sylhet was designated ST 26A, and the brick soling of shorter sections in Sylhet and Faridpur was designated ST 26B.

A few sections of waterbound macadam that totaled 10.77 mi in length were designated ST 41. Some of the 11.11 mi of gravel surfacing (ST 43) in Sylhet appeared to be deteriorated waterbound macadam that developed a loose surface.

The earth-surfaced roads that were included in the inventories showed a wider range of characteristics. Some that were constructed of better soils remained usable even in the wet season. Others became impassable during the wet season, although they performed well enough to carry heavy vehicles when dry. Still others could only be used for light vehicles such as jeeps even in the dry season because of narrow travelways and other deficiencies. Finally, a fourth category was determined for roads that were unsuitable for any motor vehicles. These roads were used only by pedestrians and animal traffic. The four earth roads were designated as follows:

- 45A—all vehicles, all seasons;
- 45B—all vehicles, 7 months a year;
- 45C—jeeps only; and
- 45D—carts and pedestrians only.

All pavement shoulders were constructed of earth, except for a short road section in the Rangpur district that had a shoulder of brick soling. In some cases, short and narrow strips of HBB had been placed on the shoulder, but this appeared to serve primarily to widen curves. Earth shoulders were designated Shoulder Type 30. All surface and shoulder types and their identification codes are listed in Table 1.

The road types in each of the districts are summarized in Table 2. Of the total roadway network in the three districts, 71 percent consist of earth roads, 2 percent are gravel, brick, cobble, or macadam, and about 27 percent are surfaced (about 16 percent bituminous and 8 percent HBB).

IDENTIFICATION OF MAINTENANCE ACTIVITIES

It is useful to list roadway elements as a step in identifying the type of maintenance activities that must be performed. The maintenance activities that were identified for the roads in Zila are listed as follows:

Pavements
Portland cement concrete
Bituminous asphalt
Herringbone brick
Single-layer cobble or brick
Shoulders Along Pavements
Herringbone brick
Soling only; brick or other
Earth
Unpaved Roadways
Waterbound macadam
Gravel or other selected materials
Earth
Drainage
Primary ditches (longitudinal)
Secondary ditches (crown, intercept, and bleeder)
Culverts (ring, box, arch, and multiple box)
Culvert channels
Bridge channels

TABLE 1 ZILA ROADS SURFACE MAINTENANCE CLASSIFICATIONS

Type	Code	Surface Description
21	RP	Portland cement cncrete pavement
22A	BIT	Bituminous pavement; good condition
22B	BIT	Bituminous pavement; poor condition
25	HBB	Herringbone brick pavement
26A	SOL	Single-layer cobble pavement
26B	SOL	Single-layer brick pavement
41	WBM	Waterbound macadam
43	GRV	Gravel or other selected material surface
45A	EA	Earth; all vehicles, all seasons
45B	EA	Earth; all vehicles, 7 mo/yr
45C	EA	Earth; jeep only (no trucks)
45D	EA	Earth; cart and pedestrian only
Shoulder Classifications		
25	HBB	Herringbone brick
26A	SOL	Soling only; brick or other
30	EA	Earth

TABLE 2 SUMMARY OF ROAD TYPES BY DISTRICT

		Number of Miles By District			
Code	Surface Type	Rangpur	Sylhet	Faridpur	Total
21	Portland cement concrete pavement	16.09	6.21	3.07	25.37
22A	Bituminous pavement; good condition	31.88	28.01	49.36	109.25
22B	Bituminous pavement; poor condition	20.90	43.80	1.06	65.76
25	Herringbone brick pavement	65.50	0.19	24.07	89.76
26A	Single-layer cobble pavement	0	2.33	0	2.33
26B	Single-layer brick pavement	0	0.28	0.20	0.48
41	Waterbound macadam	4.31	5.84	0.62	10.77
43	Gravel or other selected material surface	0	11.11	0	11.11
45A	Earth; all vehicles, all seasons	230.49	46.05	0	276.54
45B	Earth; all vehicles, 7 mo/yr	115.90	13.74	52.96	182.60
45C	Earth; jeep only (no trucks)	46.53	2.70	74.72	123.95
45D	Earth; cart and pedestrian only	109.76	14.77	71.90	196.43
Total		641.36	175.03	277.96	1,094.35

Right-of-Way
Travel way
Embankment slopes
Cut slopes
Borrow Areas
Structures
Bridges
Retaining walls and bank protection
Traffic Control Devices
Signs (controls, warning, and information)
Signals (electric or manual)
Railroad crossing protection
Pavement markings
Guardrail (not on bridges)
Edge and curve markers; mileposts

The maintenance activities are grouped under the general categories of pavements, shoulders along pavements, unpaved roadways, drainage, right-of-way, structures, and traffic control devices. By analyzing this list of elements, it is possible to calculate the problems that can occur in each activity and the actions that are necessary to resolve them in general terms. Examples of maintenance activities in response to adverse conditions on certain highway elements are shown in Table 3.

PERFORMANCE STANDARDS

After all maintenance activities that might be required on various Zila roads were identified, the next step was to decide how each activity should be performed. Decisions had to be made on which procedures to use, which tasks had to be completed, what the best crew size was, what materials and equipment were needed, how the finished work should be measured, and how much the crew should complete in a day. These questions all related to how the work was to be performed. The answers were recorded on a single form for each activity. The completed form then became a Performance Standard.

TABLE 3 EXAMPLES OF MAINTENANCE RESPONSES

Condition of Road Element	Maintenance Activity
Pavements	
Serious spalls and broken areas in portland cement concrete	Clean and patch
Potholes in bituminous pavement	Trim and patch
Surface wear, surface cracking, or oxidation of bituminous pavement	Seal coat
Local base failure in any pavement	Remove and replace base; patch surface
Moderate deformations in bituminous pavement	Place level course or overlay
Abrupt settlements in any pavement	Excavate and repair base; patch surface
Serious wear or breakage in brick surface	Remove and replace worn brick
Extensive surface failure on short sections of rigid or flexible pavement	Overlay with bitumen mix
Extensive surface or base failure on long sections of any pavement	Advise higher office for reconstruction
Unpaved Roadways	
Potholes or short raveled sections in waterbound macadam	Patch or repair with similar materials
Roughness or potholes in gravel or other selected material roadways	Grade or fill low areas with similar material; grade to drain and compact
Thin gravel or WBM surfacing rutting through	Replace lost surfacing, grade to drain, and compact
Rutted, settled, or deformed earth surface	Grade to drain
Muddy, unstable earth surface	Drain if possible
Narrow roadway (embankment sloughed)	Restore width
Encroaching vegetation	Trim, cut, or remove to restore width; grade to drain.
Paved Shoulders	
For similar conditions, the same maintenance procedures apply as for other pavements	

Not all maintenance activities that were identified as necessary had an established Performance Standard. Some activities, such as bridge maintenance, were so variable that the work could not be simply described. All activities had to have an identifying number. This number was used to estimate and budget needed work, and to report the costs of work, labor, equipment, and materials.

Performance Standards were written only for basic, repetitive activities. A total of 22 Performance Standards were developed in the initial maintenance plan. A typical example of a Performance Standard is shown in Figure 3. They detailed, for example, how to regrade an earth road or shoulder and how to patch the surface of a gravel or pavement. The Performance Standards were based on preliminary findings from the pilot maintenance work that was performed on the project; they will be verified, adjusted, and revised as the maintenance program proceeds.

MAINTENANCE LEVELS

Modest maintenance levels were established for the Zila roads for two major reasons. First, new sources of funds will have to be found by the government to initiate the work program. Second, the roads are local by definition and carry modest volumes of traffic. Proposed maintenance levels are listed in Figure 4. The amount of activity required to perform these routine and periodic tasks is expressed as a percentage of the road surface area, amount per mile, repetitions per year, or cycle of years. The estimated maintenance levels must be reviewed each year and adjusted to actual experience.

In regions in which the climate is dry and the traffic volume is low, seal coating may only be needed at intervals as long as 8 to 10 years if the bituminous pavements are well constructed with good materials and the patches are sealed. Unfortunately, these conditions do not apply to Zila roads and sealing must therefore be performed every 5 years on 22A pavements and every 3 years on 22B pavements. The patching rates shown for all types of surfaces were intended to reflect low maintenance levels. It was assumed that the road surface would have deteriorated significantly before it was repaired. The low percentage rate for the regrading or recambering of gravel roads was based on experience in Sylhet, the only district with gravel roads. These roads held their shape very well. For all categories of earth roads, the regrading rates in Sylhet are less than those for Rangpur and Faridpur because the condition of soil was better in Sylhet. The reshaping of earth shoulders along paved roads is set at a higher rate than for earth roads because they are often damaged by vehicles and other traffic. It is also important that they be in usable condition and shaped to drain water away from the pavement.

WORKLOAD

Data sheets were made from the Basic Road List (see Figure 2) for each maintenance group. The data sheets listed the individual road sections and their corresponding surface type

ZILA ROADS MAINTENANCE
AND IMPROVEMENT PROJECT

PERFORMANCE STANDARD

ACTIVITY NAME	NUMBER	WORK UNIT
Patch surface on bituminous pavement	22.02	Square foot (sft)

DESCRIPTION AND PURPOSE

Patching of failed areas in bituminous pavements (or similar paved shoulders) using bitumen premix or other materials to repair or correct potholes, settlements, edge breakage, ravelling and other deficiencies.

MAINTENANCE CRITERIA

CONDITION

Vertical difference of 1" or more in travelway (or shoulder). Well-defined holes or depressions which require filling and leveling.

OBJECTIVE

Restore the integrity and smoothness of the pavement.

PERSONNEL

NOS:HRS	CODE
1:4 Work Assistant	WKA
1:8 Driver	TDR
1:8 Gang Leader	GAL
5:8 Labourers	LAB

EQUIPMENT

NOS:HRS	CODE
1:8 Flatbed Truck	02
1:8 Vibrating plate tamper	05

AVERAGE QUANTITIES

MATERIALS PER DAY	CODE
42 cft (loose) bitumen premix	PRMX
20 lbs. 80-100 bitumen (tack)	BITU

DAILY PRODUCTION

200 sft

MATLS. PER WORK UNIT	CODE
0.208 cft (loose) bitumen premix	PRMX
0.100 lb. 80-100 bitumen (tack)	BITU

YIELD RATE

3.333 sft per man-hour

DATE	DATE REVISED
December 1, 1984	

FIGURE 3 Typical Performance Standard and instruction sheet.

METHOD AND PROCEDURE

NUMBER 22.02

1. Place warning signs or flagmen.

2. Trim and shape hole to be patched, using pick and shovel. Excavate to solid, stable material on bottom and cut near vertical sides around the outside of the failure, with a minimum depth of 1-1/2". It is not necessary or desirable to give the hole a rectangular shape. Clean out all loose and broken material with shovel and broom. For depressions, clean surface well. The surfaces must be dry. Do not block shoulder or ditch drainage with material removed.

3. Paint the sides and bottom of the hole very lightly with heated tack coat, covering not more than 50% of the area. Tack surface of settled areas in the same manner.

4. Fill hole or depression with bitumen premix, raking the material to get a uniform density and a level surface just high enough to allow for compacting the patch to the same height as the surrounding pavement. On large patches, use string line or screed board to ensure a smooth surface.

5. Compact the mix thoroughly in the hole with vibrating plate and hand tamper, or hand tamper alone. The finished patch should be level with the adjacent pavement or up to 1-1/2" higher, but never lower. If dry aggregate is used for a macadam patch, it should be thoroughly compacted to final shape before applying bitumen and fine aggregate.

6. After completing all patching in a work area, remove warning signs or flagmen.

TO BE REPORTED

1. The work location, by road route and section.

2. All labour and equipment hours, and the materials actually used, in pounds of tack coat and cubic feet of bitumen premix (loose measure), or cubic feet of dry aggregate plus pounds of bitumen for macadam patching.

3. The work completed each day, in square feet, measured.

FIGURE 3 continued.

classification. The lengths and widths gathered from the inventories were entered on the data sheets and used to compute surface and shoulder areas. Information on vegetation control, lengths of ditches, volumes of culvert cleaning, and number of bridges were added to the sheet. A typical data sheet is shown in Figure 5.

The information for each maintenance group was summarized on a Workload Base Sheet, which combined all groups of the same surface type, and other work quantities. These sheets represented the total areas and other quantities to which activities were to be applied according to the maintenance level that was established. A typical Workload Base Sheet is shown in Figure 6.

The maintenance group workload, or number of crew-days for each standard activity, is the base quantity of the element times the annual rate of the maintenance level, divided by the daily production figure of the Performance Standard. This calculation can be simplified by arranging a worksheet with similar surface types in horizontal groups so that activities that apply to more than one type can be summed in the right-hand column, as shown in Figure 7.

RESOURCE REQUIREMENTS

When the annual workload is known, the resources needed for the program can be calculated. Resource requirements are quantified in terms of labor hours, equipment hours, and quantity of materials. The quantification of the base activities is simple because the required data are written into the Performance Standards.

Two worksheets were developed to quantify resources. One was developed to compute labor and equipment hours, and the other to quantify materials. Examples of these worksheets are shown in Figures 8 and 9. Resources also include the number of workers and pieces of equipment that must be available to do the maintenance work, as opposed to the total hours for each. Materials, of course, are the third component.

In the case of labor, adjustments were made for nonworking days, legal holidays, and leave time, as appropriate, for permanent employees. Laborers and gang leaders earned no leave or other benefits, so the required hours were converted to man days directly. A total of 245 average working days a year, or 1,960 available hours, were calculated for work assistants, roller drivers, and truck drivers. Equipment availability was adjusted by a factor of 75 percent to reflect servicing and repair, which reduced the assumed number of available hours a year to 1,758. The resource recommendations that were developed from these computations are shown in Figure 10. Casual labor hours and material quantities were omitted from these sheets because it was assumed that such resources would be provided as required. The conversion to total numbers of men and machines provided the basis for staffing and equipment allocations to the maintenance groups in the three districts.

Although it was intended that the recommended maintenance methods for Zila roads be highly labor-oriented and equipment

Shown as Percent of Road Surface Area, Amount per Mile, Repetitions per Year or Cycle of Years Between Repetitions

ACTIVITY	UNIT	21 P C C Pavement	22A Bit. Pavement Good Condition	22B Bit. Pavement Poor Condition	25 H B B Pavement	26A / 26B Single Pavement: Cobble / Soling	41 W B Macadam Surface	43 Gravel Surfacing	45A Earth--All Vehicles, All Seasons	45B Earth--All Vehicles 7 Months Per Year	45C Earth--Jeep Only (No Trucks)	45D Earth--Cart and Pedestrian Only
		QUANTITY										
21.03 Repair PCC with Bit. Mix	% Surface	0.75										
22.02 Patch Bit. Surface	% Surface		0.75	1.50								
22.03 Repair Base and Surface	% Surface	0.25	0.25	0.50								
22.04 Bituminous Overlay	% Surface			0.25								
22.05 Seal Small Areas	% Surface	1.25	1.25	2.50								
22.06 Seal Coat (Periodic)	Cycle-Years		5	3								
25.01 Patch Surface H B B	% Surface				0.25							
25.02 Patch Base and Surface H B B	% Surface				0.25							
26.01 Patch Soling or Cobble Pavement	% Surface					0.50						
30.01 Re-grade Earth Shoulders	% Surface	75	75	75	75	75	75	75				
41.01 Patch W B Macadam Surface	% Surface						0.50					
41.02 Replace W B M Surface (Periodic)	Cycle-Years						8					
43.01 Patch Gravel Surface	% Surface							0.80				
43.02 Replace Gravel Surface (Periodic)	Cycle-Years							5				
43.03 Re-grade Gravel Surface	% Surface							10				
45.01 Patch Earth Surface	% Surface								1.00	1.00	0.50	0.25
45.02 Re-grade Earth Surface — Sylhet	% Surface								20	20	10	5
45.02 Re-grade Earth Surface — Other Districts	% Surface								50	50	25	10
50.01 Clean Ditches	rft/mile	40	40	40	40	40	40	40	40	40	40	20
50.02 Clean Culverts	cft/mile	10	10	10	10	10	10	10	10	10	10	5
60.01 Trim Weeds or Brush	Repetitions	1	1	1	1	1	1	1	1	1	1	1

FIGURE 4 Maintenance levels and annual amounts sheet.

Road Identification and
Base Working Quantities

Units: Length-miles
Width-feet
Area-000sft

District: RANGPUR

Maint. Area: Nilphamari

Date: Nov. 10, 1984

ROAD NO. & SECTION	LENGTH (Miles)	SURFACE TYPE	SURFACE WIDTH	SURFACE AREA	SHOULDER TYPE	SHOULDER TOTAL WIDTH	SHOULDER TOTAL AREA	VEGETATION sft/ MILE	VEGETATION TOTAL AREA	DITCH rft/ MILE	DITCH TOTAL AREA	CULVERT cft/ MILE	CULVERT TOTAL cft	NO. OF BRIDGES
RNI-4A	1.96	45A	10.4	107.63										
B	0.28	21	10.0	14.78	30	15.4	22.77							
C	3.20	45A	23.0	388.61										
D	2.73	45A	18.6	268.11										
E		R & H SYSTEM												
F	0.12	25	10.5	6.65	30	10.8	6.84							
G	0.37	21	10.0	19.54	30	11.2	21.88							
H	0.19	25	10.0	10.03	30	13	13.04	0.6	18.64	40	1242	20	621	10
I	0.16	21	14.0	11.83	30	12.8	10.81							
J	0.03	25	10.0	1.58	30	12	1.90							
K	0.34	21	10.0	17.95	30	17.1	30.70							
L	0.96	25	10.0	50.69	30	17.9	90.73							
M	4.13	45A	25.2	549.52										
N	16.59	45A	26.7	2338.79										
O	2.11	45C	14.3	159.31				0.3	0.63	20	42	10	21	
RNI-11A	7.49	45D	15.8	624.84										
B	3.67	45D	15.7	304.23				0.3	3.87	20	258	10	129	5
C	1.74	45D	19.2	176.39										
RNI-12A	0.03	25	10.0	1.58	30	14	2.22							
B	0.68	21	14.0	50.26	30	12	43.08	0.6	6.22	40	415	20	207	5
C	4.22	45A	15.4	343.14										
D	5.44	45A	19.4	557.23										

FIGURE 5 Road inventory data sheet.

Road Surface Areas and Other Quantities by Maintenance Areas

SURFACE AREAS, OR OTHER QUANTITIES	UNITS	NILPHAMARI	KURIGRAM AND LALMONIRHAT	RANGPUR	GAIBANDHA
Type 21	000 sft	147.57	51.32	316.57	404.29
22A		66.66	194.31	1313.16	194.20
22B		282.70	0	417.08	404.95
25		1757.73	299.70	1564.86	357.39
26A		0	0	0	0
26B		0	0	0	0
41		1.55	0	239.08	0
43		0	0	0	0
45A		10,393.52	1839.45	7483.20	4545.40
45B		357.24	0	10,108.05	1313.09
45C		159.31	0	4276.72	0
45D		3869.83	503.50	4311.47	822.19
30 Shoulder		2235.37	520.09	3810.53	1125.49
26B Shoulder		0	0	0	45.95
Vegetation Clearing		94.79	17.63	1154.74	52.30
Ditch Cleaning	rft	6317	1176	11,547	3488
Culvert Cleaning	cft	3158	588	5775	1754
Bridges	nos	58	5	119	37

FIGURE 6 Workload base sheet—Rangpur district.

use be kept to a minimum, it was the consultant's opinion that some power equipment was necessary and warranted. Compaction equipment in small and medium sizes is considered essential for good results in some maintenance and construction activities. Trucks are also desirable to carry fairly large amounts of loose or sticky materials to various work sites.

Staffing requirements that were derived from the analyses indicated that several maintenance groups did not have a large enough maintenance workload to become separate organizations with their own staff and facilities. Some inconsistencies were also found between the initial equipment acquisition program and equipment requirements for the recommended maintenance program. There were not enough flatbed trucks, but more than enough four-by-four pickup trucks. A serious shortage of rollers was also indicated. The number of vibrating plate compactors was exact; six were called for and six were acquired.

It was recommended that some additional equipment that was not included in the current acquisition program be made available to the Zila road maintenance program. Overall, it was found that the resource needs that were quantified in the proposed maintenance program were reasonable and practical. Required staffing levels for permanent employees generally conformed to the organizational charts proposed by the local government's engineering bureau for the pilot maintenance program.

BUDGETING

Once unit rates were developed or adopted for the three basic cost elements, they were applied to the quantified resource needs of each maintenance group on a Cost Calculation Sheet (see Figure 11). The following assumptions were made:

- At least one work assistant was assigned to each maintenance group even if the quantified hours were less than 1,960.
- If the annual hours were above 750, a full-time position was assumed for roller drivers and truck drivers. If the annual hours were less than about 700, it was assumed that drivers would be hired temporarily when required.
- Emergency maintenance was assumed to require labor hours that totaled about 1 percent of regular labor or personnel costs.

An example of a proposed road maintenance budget for a typical maintenance group is shown in Figure 12.

PROGRAMMING AND SCHEDULING

The annual programs of routine maintenance were derived directly from the rounded crew-days for each activity, as shown on the Calculation of Resource Needs sheets (see Figure 8).

Program Year: 1985/86 District: SYLHET
Date: DEC. 4, 1984 Maint. Area.: SYLHET SADAR

MAINT. ACTIVITY	DAILY PROD.	MAINT. LEVEL	WORK QUANTITY	CREW DAYS	MAINT. LEVEL	WORK QUANTITY	CREW DAYS	MAINT. LEVEL	WORK QUANTITY	CREW DAYS	MAINT. LEVEL	WORK QUANTITY	CREW DAYS	TOTAL CREW DAYS
			Surface Type 22A 6,52,160 sft			Surface Type 22B 10,26,520 sft			Surface Type 26A 57,020 sft			Type 30 Shoulder 14,39,970 sft		
22.02	200	0.75%	4,891	24.4	1.50%	15,398	77.0							101.4
22.03	100	0.25%	1,630	16.3	0.50%	5,133	51.3							67.6
22.04	600				0.25%	2,566	4.3							4.3
22.05	600	1.25%	8,152	13.6	2.50%	25,663	42.8							56.4
22.06	1,800	20%	1,30,432	72.5	33.3%	3,41,831	189.9							262.4
26.01	80							0.50%	285	3.6				3.6
30.01	2,000										75%	10,79,978		540.0
			Surface Type 41 2,960 sft			Surface Type 43 3,98,340 sft								
41.01	300	0.50%	15	0.1										0.1
41.02	100	12.5%	370	3.7										3.7
43.01	350				0.80%	3,187	9.1							9.1
43.02	200				20%	79,668	398.3							398.3
43.03	4,000				10%	39,834	10.0							10.0
			Surface Type 45 11,54,870 sft			Surface Type 45B 3,16,900 sft			Surface Type 45C 1,14,050 sft			Surface Type 45D 1,77,540 sft		
45.01	1,000	1.00%	11,549	11.5	1.00%	3,169	3.2	0.50%	570	0.6	0.25%	444	0.4	15.7
45.02	10,000	20%	2,30,974	23.1	20%	63,380	6.3	10%	11,405	1.1	5%	8,877	0.9	31.4
			Ditch Cleaning											
50.01	500	100%	62,762 rft	125.5										125.5
			Culvert Cleaning											
50.02	120	100%	1,005 cft	8.4										8.4
			Vegetation Clearing											
60.01	2,000	100%	2,74,920 sft	137.5										137.5
71.09	General	Bridge Maintenance												
	Annual	man-hours = No. Bridges			x 10.5									
			= 26 x 10.5 =		272 m.h.									

FIGURE 7 Crew-day calculation sheet.

Listed in the second column are the estimated number of accrued days of the activity that will be necessary during the year if it is performed in compliance with the performance standards. The work programs have to be issued in a way that they can be understood and communicated at all levels. The method that appeared to work best to initiate maintenance in the Zila road districts was to issue program cards. One card was made for each repetition of each activity called for in the program.

The number of cards issued represented the number of days the work was expected to be performed during the year. The cards were created at the beginning of the program year, and should have been accounted for by serial numbers or logs to ensure that work performed had conformed to the program and budget. One advantage of this method was that the same cards that were used to record the work at the time it was done were used to report information needed for control and evaluation. The same cards can also be used for basic payroll and equipment usage data in some accounting systems. The recommended format for the card is shown in Figure 13, in which a sample card is filled out that indicates the status after a day's work is completed.

The field office work schedule ensures that the work will get done and the work force will be occupied most of the time. Although some jobs must be performed at fairly regular intervals throughout the year, others can be done when the

Program Year: 1985/86 District: RANGPUR

Date: DEC. 11, 1984 Maint. Area: NILPHAMARI

ACTIVITY	TOTAL CREW DAYS	PERSONNEL										EQUIPMENT							
		WKA HOURS		RDR HOURS		TDR HOURS		GAL HOURS		LAB HOURS		01 HOURS		02 HOURS		03 HOURS		05 HOURS	
		PER C-D	TOTAL	PER C-D	TOTAL	PER C-D	TOTAL	PER C-D	TOTAL	PER C-D	TOTAL	PER C-D	TOTAL	PER C-D	TOTAL	PER C-D	TOTAL	PER C-D	TOTAL
21.03	6	4	24	-		8	48	8	48	40	240	-		8	48	-		8	48
22.02	24	4	96	-		8	192	8	192	40	960	-		8	192	-		8	192
22.03	20	4	80	-		8	160	8	160	40	800	-		8	160	-		8	160
22.04	2	8	16	4	8	8	16	8	16	56	112	4	8	8	16	-		-	
22.05	17	4	68	-		8	136	8	136	32	544	-		-		8	136	8	136
22.06	60	8	480	8	480	8	480	8	480	72	4,320	8	480	8	480	-		-	
25.01	44	4	176	-		2	88	8	352	32	1,408	-		2	88	-		-	
25.02	110	4	440	-		2	220	8	880	24	2,640	-		2	220	-		-	
26.01		4		-		8		8		24		-		8		-		-	
30.01	839	4	3,356	-		-		8	6,712	56	46,984	-		-		-		-	
41.01	1	4	4	-		8	8	8	8	40	40	-		-		-	8	8	8
41.02	2	4	8	8	16	8	16	8	16	40	80	8	16	8	16	-		-	
43.01		4		-		8		8		40		-		8		-		-	
43.02		4		8		8		8		40		8		8		-		-	
43.03		4		8		8		8		56		8		8		-		-	
45.01	118	2	236	-		-		8	944	32	3,776	-		-		-		-	
45.02	581	4	2,324	-		-		8	4,648	56	32,536	-		-		-		-	
50.01	13	2	26	-		-		8	104	40	520	-		-		-		-	
50.02	27	2	54	-		-		8	216	40	1,080	-		-		-		-	
60.01	48	2	96	-		-		8	384	40	1,920	-		-		-		-	
82.01	6	8	48	-		-		8	48	16	96	-		-		-		-	
82.02	18	8	144	-		-		8	144	32	576	-		-		-		-	
	TOTAL		7,676		504		1,364		15,488		98,632		504		1,220		144		544

FIGURE 8 Calculation of resource needs sheet.

Program Year: 1985/96 District: RANGPUR
Date: DEC. 11, 1984 Maint. Area: NILPHAMARI

ACTIVITY	TOTAL CREW DAYS	BITU (lb) PER C-D	BITU (lb) TOTAL	KERO (gal) PER C-D	KERO (gal) TOTAL	MC 25 (lb) PER C-D	MC 25 (lb) TOTAL	PRMX (cft) PER C-D	PRMX (cft) TOTAL	BASE (cft) PER C-D	BASE (cft) TOTAL	JHAM (nos) PER C-D	JHAM (nos) TOTAL	BRST (cft) PER C-D	BRST (cft) TOTAL	SCRN (cft) PER C-D	SCRN (cft) TOTAL	SND3 (cft) PER C-D	SND3 (cft) TOTAL	SND 5(cft) PER C-D	SND 5(cft) TOTAL TOTAL
21.03	6	20	120					42	252												
22.02	24	20	480					42	1,008												
22.03	20	10	200					21	420	63	1,260										
22.04	2	60	120					125	250												
22.05	17	60	1,020															9	153		
22.06	60	180	10,800															27	1,620		
25.01	44											500	22,000							6	264
25.02	110									18	1,980	312	34,320							5	550
26.01										37		224								2	
41.01	1													32	32	6	6				
41.02	2													125	,250	25	50				
43.01										88											
43.02										230											
43.03										50											
Subtotal	PRMX								1,930												
82.01	6	1,350	8,100	60	360																
82.02	18					567	10,206							108	1,944						
	TOTAL		20,840		360		10,206				3,240		56,320		2,226		56		1,773		814

FIGURE 9 Calculation of resource needs and materials sheet.

Program Year: 1985/86 District: Rangpur
Date: Dec. 21, 1984 Maint. Area: Nilphamari

Description	Quantified Hours	Equivalent Unit Days	Equivalent Units[a]	Recommendation
Work Assistant	7,676	959.5	3.9	Staff 4 WKR
Roller Driver	504	63.0	0.3	Temporary hire, or share
Truck Driver	1,364	170.5	0.7	Staff 1 TDR
Steel-Wheel Roller	504	63.0	0.3	Rent, or share
Flatbed Truck	1,220	152.5	0.7	Assign 1 truck
4 x 4 Pickup	144	18.0	0.1	Substitute
Vibrating Plate Tamper	544	68.0	0.3	Rent, share or substitute

[a] At 1,960 available hours per year for personnel, and 1,758 hours per year for machines.

FIGURE 10 Resource recommendations sheet.

Program Year: 1985/86 District: RANGPUR
Date: DEC. 23, 1984 Maint. Area: NILPHAMARI

LABOUR

TITLE	QUANTITY	RATE, TK	ANNUAL AMOUNT, TK
Work Assistant	4 men	17,117 per year	68,468
Roller Driver	504 m-h	8.3 per m-h	4,400
Truck Driver	1 man	17,117 per year	17,117
Gang Leader	1,936 m-d	35 per m-d	67,760
Day Labourer	12,329 m-d	30 per m-d	3,69,870
Bridge Labour	1,450 m-h	4.05 per m-h	5,873
		Subtotal	5,33,488
Emergency Maint.	Lump Sum	1.0%	5,335
		SUBTOTAL LABOUR	5,38,823

EQUIPMENT

TYPE	HOURS	RATE, TK	ANNUAL AMOUNT, TK
01 Roller	504	212	1,06,848
02 Flatbed Truck	1,220	231	2,81,820
03 4 x 4 Pickup	144	86	12,384
05 Plate Tamper	544	21	11,424
		Subtotal	4,12,476
Emergency Maint.	Lump Sum	1.0%	4,125
		SUBTOTAL EQUIPMENT	4,16,601

MATERIAL

ITEM	QUANTITY/UNIT	PRICE, TK	ANNUAL AMOUNT, TK
Bitumen	20,840 lb	4.50	93,780
Kerosene	360 gal	40	14,400
Granular Base	3,240 cft	15.50	50,220
Jhama Brick	56,320 nos	1.50	84,480
Broken Stone (3/4")	2,226 cft	20.00	44,520
Screening	56 cft	6.00	336
Sand 3 (F.M. 1.0-1.5)	1,773 cft	5.00	8,865
Sand 5 (F.M. 0.5-0.8)	814 cft	3.00	2,442
		Subtotal	2,99,043
Emergency Maint.	Lump Sum	1.0%	2,990
		SUBTOTAL MATERIAL	3,02,033
		GRAND TOTAL	12,57,457

FIGURE 11 Cost calculation sheet.

Date: Dec. 31, 1984 District: Rangpur
Maint. Area: Nilphamari

Major Object of Expenditure	Actual Previous Year	Estimated Current Year	Proposed Budget
Personal Services and Related Benefits			TK 5,38,823
Contractual Services			
Materials and Supplies			3,02,033
Equipment Cost			4,16,601
Capital Outlay			
Total Direct Expenditures			TK 12,57,457

FIGURE 12 Proposed 1985/86 district road maintenance budget sheet.

Activity No.: 60.01 Name: TRIM WEEDS AND BRUSH

Date of Work: 10 SEPT 85 Maint. Area: RANGPUR - NILPHMARI

Route No. RNI-35 Sect. D Card No. 04 Total 85/86 48

RESOURCES USED

LABOR				EQUIPMENT		
CODE	HOURS	NAMES	STD CREW ___	CODE	ID NUMBER	HOURS
WKA	3	Mr. Anthony				
GAL	8	Abdul Gani			None	
LAB	8	Jalil				
	8	Rashid				
	8	Ismed				
	8	Moin				
	8	Abdul				

MATERIALS

CODE	DESCRIPTION	QUANTITY	UNIT
	None		

WORK COMPLETED

Unit of Work Sft Amount Done 1,850

FIGURE 13 Program and report of work card.

opportunity arises. Some activities, such as seal coating, require dry conditions. Alternate activities for moderately wet conditions could be the repair of brick roads and shoulders, brush cutting, and breaking and hauling aggregates over hard-surfaced roads. Ditches can also be cleaned while the drainage is being checked during rainy weather.

Culvert repairs or channel work can only be performed when the water is low. Warm, dry conditions are required when bitumen is being used in major activities. These conditions were considered when the work schedules for the proposed maintenance activities were developed. However, it was recognized that maintenance is not suited to close and rigid scheduling, but must be flexible enough to allow for unexpected work.

STATUS REPORTS

Reporting the status of work accomplished is almost automatic with the use of the Program and Report of Work Card. No other forms are needed to record work, although some types of summary and analysis forms could be useful. It is important that the report of work completed be reasonably accurate, and that the work be measured in the same manner and the same units indicated on the Performance Standards for that activity.

The information that is reported back to headquarters, and gathered through inspections, must be analyzed to answer the broad questions of whether or not the management system is ensuring that roads are adequately maintained, and whether or not it can be improved.

It is anticipated that the classification of surface types will be reviewed periodically to ensure that it covers all required cases and to determine if any types can be eliminated or combined to simplify the list. The activities list can be changed whenever it is found that another category is needed, or that one of the major series should be deleted.

When enough information is accumulated at the end of the maintenance year, the Performance Standards should be compared with reports of how the work was actually performed and its results. All elements of the standards must be checked, including the resources and procedures that were employed and the production rate. Standards should be adjusted, especially if different materials and new equipment are substituted.

Another crucial evaluation is whether or not the assumptions on maintenance levels were satisfactory. The first question is whether or not all programmed maintenance work was performed. The second is whether or not that amount of work was enough or more than necessary. This evaluation should be made through inspections and judgments to determine if the conditions of the different road elements were preserved or if the roads have deteriorated. The maintenance levels can then be adjusted according to the requirements of the roads and the resources available. It is anticipated that these activities will be undertaken in an extension of the project, the paperwork for which is now under way.

CONCLUSION

An outline was presented of the maintenance management plan that was developed and initiated in Bangladesh on the Zila local road system. The various steps of the plan were detailed. The maintenance management system is in place and it appears that the consultant will have the opportunity to evaluate the results and participate in desirable changes and revisions.

Now that the system has been installed with successful initial results, it appears that a regular, periodic maintenance program can be institutionalized for the Zila road system in Bangladesh. It is anticipated that the system will be extended gradually to all other districts throughout the country.

A Periodic Maintenance Management System for Low-Volume Roads in Niger

PETER LONG, GONI LAWAN, AND PASCAL MIGNEREY

The establishment of a management system for preparing periodic maintenance programs in Niger is discussed. The Ministry of Public Works in Niger realized that a rational system was needed to develop a maintenance program to safeguard the investment in the road system. This system would have to be tailored to Niger, a poor country with a sparse but expansive road network and an extremely limited number of engineers and technicians. A Road Maintenance Unit was therefore established with the objectives of monitoring the condition of the network and the volume of traffic using it, and developing periodic maintenance programs based on this evaluation. A description is provided of the methods and techniques that were used to accomplish these objectives.

The establishment of a management system for preparing periodic maintenance programs and budgets in Niger is discussed. Niger is a landlocked country in West Africa that extends from the northern fringe of a rain-fed agricultural region into the heart of the Sahara desert. The population of about 6 million is growing at a rate of 3 percent a year. Nearly 90 percent of the population is concentrated along the southern border in the 12 percent of the land that is arable. It is one of the poorest countries in the world with a per capita GNP of about $200, and an economy dominated by subsistence agriculture. Adequate transportation is critical to Niger. Large stretches of nearly empty land exist between population centers. High transportation costs limit the competitiveness of Niger's products on the world market, make imported items expensive, and add to the cost of delivering services to distant urban centers and the scattered rural population.

The road network in Niger is 9800 km long; 3160 km of the roads are paved. Traffic volumes are low; outside urban areas, the highest traffic volume is less than 1,000 vpd, and only 546 km or 17 percent of the paved roads carry more than 300 vpd.

The road network is relatively new; the first one-lane paved roads were built in the 1960s. In the 1970s, all of these roads were widened to a two-lane width. A two-lane standard is now employed throughout the network. The paved roads are therefore at a point at which extensive periodic maintenance is required for the first time. The Public Works Department (PWD) grew into existence over the same 25-yr period. The Ministry of Public Works realized that a rational system was needed to develop a maintenance program to safeguard the investment in the road system. However, this system would have to be tailored to Niger, a poor country with a sparse but expansive road network and an extremely limited number of engineers and technicians.

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A Road Management Unit (RMU) was therefore established in the PWD with two objectives: to monitor the condition of the network and the volume of traffic using it, and to develop periodic maintenance programs for the network on the basis of this data. The RMU must perform three major tasks to accomplish these objectives: collect data, on a continuous basis, on both paved and unpaved roads; create and maintain a data bank; and develop annual medium and long-term periodic maintenance programs, including cost estimates and economic justifications. This task includes analyzing the data, making forecasts, and applying decision criteria (Figure 1).

DATA COLLECTION

The immediate goal of the RMU was to produce credible periodic maintenance programs. In view of the limited human and financial resources available, data collection was confined to the parameters that were necessary to achieve this goal. The RMU and PWD are, however, dynamic organizations and additional tasks will be added in the future when interest in them becomes apparent.

Some data, called office data, can be collected without field visits, such as the definition of the network, historical information on each road link, and quarry locations. In Niger, the PWD is officially responsible only for "classified" or national highways. However, partly because the PWD is the only organization with maintenance capabilities, and partly because the official classification system is out of date, the PWD actually maintains more than the classified network. The maintained network, therefore, includes paved and unpaved national, regional, and rural roads, all of which are managed by the PWD. A reference system of nodes and links was defined for this network. Each link is homogeneous as much as possible from the point of view of traffic and class of road.

Other office data include the history of each link in the network, such as the dates of original construction, major resurfacing, and strengthening; quarry locations; and quality and quantity of available materials. Long stretches of road in Niger are far from suitable road building materials and transport is a major element of maintenance costs.

The collection of data by field survey was not the responsibility of any existing PWD unit. The RMU therefore established, trained, and administered its own survey crews, three for traffic counts and one for road inventory and condition surveys.

Traffic Counts

Traffic data are collected by manual and automatic counts by three crews based in regional centers. The count program is in operation 11 months each year. The manual counts last for either 3 or 7 days; the 3-day counts include a local market day. These counts include a classification of vehicles into five types.

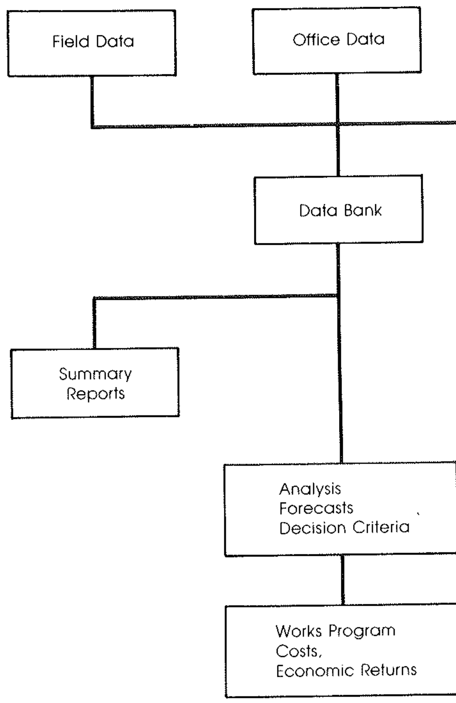

FIGURE 1 Functions of the Niger Road Maintenance Unit.

Automatic counts are performed only on paved roads. The RMU has 12 counters, eight of which are electronic and are read by a portable microcomputer (an Epson HX20) onto cassettes. They are used with magnetic loop detectors. The other four are of an older electromechanical type and are used with pneumatic tube detectors.

Road Inventory and Condition Surveys

Pentakilometer (5-km) markers were placed on all paved roads and they are generally accurate. However, road improvements that shorten the alignment can upset the accuracy of all subsequent kilometer markers. An independent kilometer reference system was therefore established, and the existing kilometer posts were incorporated into it. Most unpaved roads do not have pentakilometer posts. Distances on these roads were measured with a precise odometer (Halda Twinmaster), and special markers were placed.

A windshield survey is a conventional, detailed visual analysis that is performed in a car and, when necessary, on foot. Distances are recorded with the precise odometer, and the roadside kilometer markers were incorporated into the measurements. The following three groups of data are recorded:

- Road characteristics, such as pavement width, right and left shoulder width, presence of village or junction location, and type of drainage structure;
- Roadside condition, such as right and left ditches, right and left shoulders, and cut or fill section; and
- Pavement condition, such as condition of surface treatment, cracking, and deformation.

The pavement condition characteristics are recorded on a scale of 0 to 5, from excellent to bad, for the surface treatment, and 0 to 3 for the extent of cracking and deformation. Data collected for unpaved roads are width, presence of corrugations, gravel loss, potholes, rutting, depressions, and crossfall.

One axle load survey has been performed using portable scales, and the RMU has also introduced in-motion weighing on an experimental basis using a piezo-electric cable. The cable is positioned in a transverse slot in the pavement, which is then filled with epoxy. The passage of an axle produces an electric signal that varies with the load, and the reading is stored in an electronic control box. The reading is calibrated using standard loads. The cable needs a stable platform for accurate measurements. Because the pavements in Niger were relatively light and flexible, the RMU decided to install the cable in the center of a 30-m-long Portland cement concrete slab. Arrangements were therefore made with the financing agency, the European Development Fund, to include the slab in an ongoing contract to widen and strengthen a section of the main east-west highway. The electronic recording system produces a histogram of the number of passing axles in five classes: 1 to 5, 5 to 9, 9 to 13, 13 to 16, and more than 16 metric tons. The legal limit for a single axle in Niger is the usual Francophone one of 13 tons, or 28,600 metric lbs. If the experimental installation is successful, similar installations might be made on other sections of the paved road network.

The French APL 25 longitudinal profile analyzer was chosen to measure road roughness. The local crew was trained in its use by a technician from the French Central Roads and Bridges Laboratory. The APL 25 records results on a standard cassette, which is directly read by a digital cassette reader for automatic input to the RMU's computer. The software, in addition to expressing the road roughness on the normal CAPL 25 scale, converts the results into the International Roughness Index scale, which is used as a standard input to the World Bank's Highway Design and Maintenance Model (HDM). Complete surveys of the paved road network in Niger were performed in 1985 and 1986.

Economic Data

Three groups of economic data are collected: the unit costs of construction, rehabilitation, and maintenance, for both periodic and routine maintenance, by contract or by force account; vehicle operating costs; and macroeconomic and traffic forecasts.

DATA INPUT AND STORAGE

The RMU was instrumental in introducing electronic data processing to the Ministry of Public Works. A list of equipment was determined in 1984 and the following items were chosen at that time:

- A Victor S1 computer with 512 kbytes of memory and two floppy disks with 1.2 Mbytes of memory,
- An Alpha 10 computer with 2 x 10 Mbyte cartridges (Bernouilli Box),
- A Hewlett Packard 7475A Plotter, and
- An Epson LQ 1500 printer.

Data are input manually by way of the keyboard, and automatically from the automatic traffic counters and the APL 25. In order to take advantage of other software now available, increase flexibility, and ease communication, the system is in the process of being updated with a new microcomputer that is IBM-compatible.

Data are stored in the following four files:

- Fixed inventory data, such as information on link start and finish, road length, pavement and shoulder width, and the presence of ditches, drainage structures, and cut or fill sections;
- Evolving inventory data, such as information on the condition of pavement surfaces, shoulders, and ditches that was collected by the windshield survey;
- Traffic data; and
- Road roughness data.

SUMMARY REPORTS ON DATA

The information in the four files can be output in summary form, including general information on each road section, summary traffic counts, and pavement condition. Data can also be output in map form on the automatic plotter. The characteristics shown by the variation in form or color of lines of the map can be representative of any parameters stored in the data bank. Data for the plotter come directly from the data bank, so the maps produced always reflect the latest input data.

DATA ANALYSIS

The first set of data was collected during 1985 and the beginning of 1986. These data were used in 1986 to develop the proposed maintenance programs for the years 1987 to 1989. Although it was originally intended that the RMU produce an annual program of periodic maintenance, the programs in Niger are funded with foreign aid. The time necessary to establish a program and the size of each program prevented the strict implementation of year-by-year programs. A 3-year program was therefore funded by several donors.

MAINTENANCE POLICIES

It was first necessary to establish a decision methodology. In 1986 the software that automatically read and input the APL roughness data was not yet available. Therefore, decision criteria were used from the three pavement condition evaluations of the windshield survey: surface treatment condition, cracking, and deformation. The first two are indications of surface distress, and the third of problems in the pavement structure below the surface. A two-dimensional decision grid was therefore developed in which increasing surface distress, defined as the sum of surface treatment condition and cracking, was plotted on the X-axis and deformation was plotted on the Y-axis. The types of maintenance intervention, which ranged from routine maintenance to strengthening the overlay and rehabilitating the road, were then related to the cells in the grid, as shown in Figure 2.

The thresholds and the different types of maintenance works in the decision grid were positioned on the basis of engineering judgment. The positioning of these thresholds was the subject of considerable discussion between engineers from the government and the World Bank. They considered engineering standards that were commonly used in Niger and West Africa, and the results of World Bank research in other parts of the world. Because the great majority of surfaces and pavements in Niger were in good condition, persons who conducted the windshield condition survey tended to rate the road severely. The grid therefore might need to be extended for use by other countries.

MAINTENANCE MODEL

A simplified version of the World Bank's HDM was developed by the Bank and consultants. The model is shown in schematic form in Figure 3. The inputs to the model are the road inventory (both fixed and variable conditions), traffic data, economic data, and the maintenance policy decision grid.

The model first adds default values for required HDM input data that are not collected in Niger, and transforms the Niger survey ratings for surface treatment condition, cracking, and deformation into HDM units. This provides the initial condition for the simulation, which is run with either a null or specified maintenance policy. The model predicts the condition of the road section at the end of the year, as a result of its initial condition and the effects of traffic and climate derived from the HDM equations that were calibrated to conditions in Niger. The predicted condition is then compared to the maintenance policies to determine if work would be required on a road section. If so, the maintenance quantities and costs for the sections are calculated. Then the updated road condition is calculated. This road condition then becomes the input for the second year of the simulation cycle, which is repeated for the total number of years of the analysis. The simulation cycle for Niger was 10 years.

The total vehicle operating costs over the road section for each year are calculated, depending on the forecasts of both traffic and road conditions. The total transport costs, which

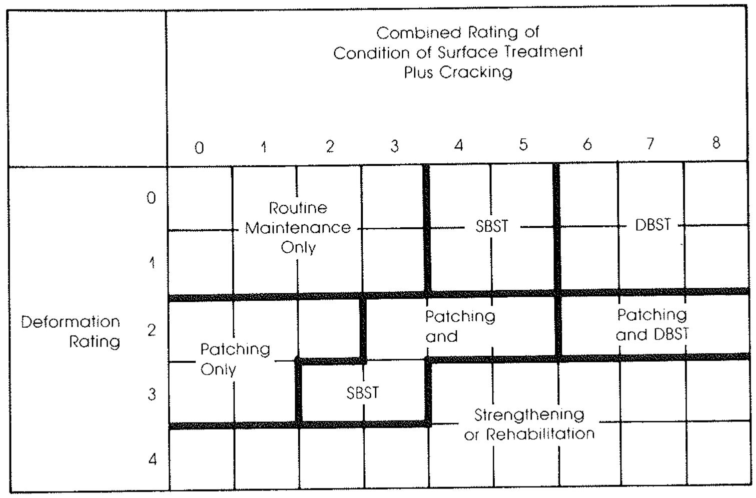

FIGURE 2 **Maintenance policy decision grid.**

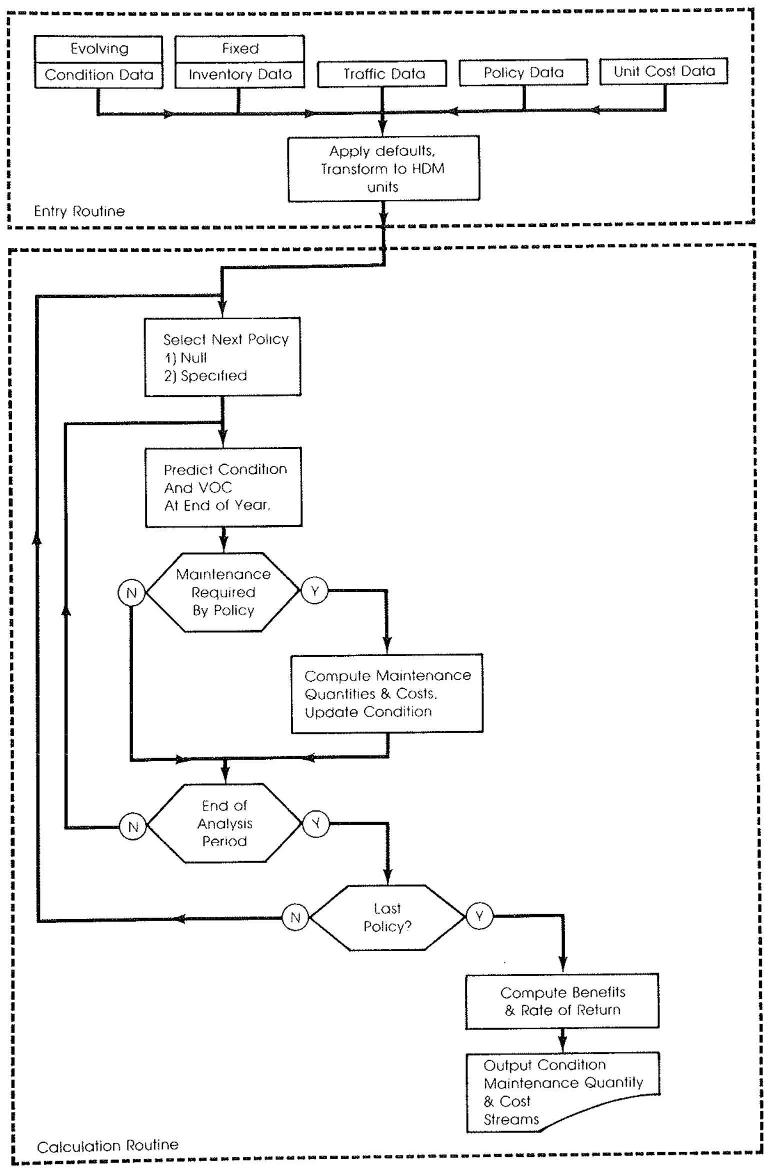

FIGURE 3 **Flow chart of computer model used to prepare maintenance program with economic indicators.**

consist of vehicle operating costs and maintenance costs, are summed for all sections in the road link. The total costs are then compared for the null and with maintenance policies. Then the costs are discounted to develop an economic rate of return for the investment in periodic maintenance on the road link.

It should be noted that this analysis is performed for each homogeneous section of a road link. In practice the type of maintenance applied can vary in roughly 100-m lengths. This level of analysis is therefore needed to produce as accurate an estimate as possible of the quantities of maintenance works.

CONTRACT DOCUMENTS

For lots that consisted of road sections that do not need to be strengthened, the results of the RMU analysis are sufficient to be directly incorporated in the bills of quantities in contract documents. However, although the average costs of improving severely deformed sections are used to permit the RMU to develop a program of works and its economic justification, the level of analysis is insufficient to define an engineering solution for construction. A conventional engineering study should be performed for such sections, including deflection testing and test pits to identify the cause of the problem, and to permit the improvement in pavement structure to be properly designed.

RESULTS OF THE ANALYSIS

The Government of Niger has as a stated policy to preserve the national heritage. The analysis performed by the RMU has been an important factor in extending this general policy to the maintenance of the road infrastructure. In spite of a financial crisis, a satisfactory level of funding has been allocated in the investment budget to periodically maintain the road network. The general policy of the government and the trend among foreign donors to lend money for road maintenance have both been strengthened by the RMU analysis. Budgetary allocations for this purpose are now supported by several donors.

FUTURE DEVELOPMENT

The maintenance program that is currently being executed was based on the decision grid of maintenance policies related to pavement condition. The next step in the program was to refine the decision grid. The vehicle for this is a national transport study that is now in progress in Niger. One important element of this study is the installation of the full HDM program in Niger. This is now possible because the HDM has been restructured to run on microcomputers with the capabilities of the IBM AT. The network data in the RMU data bank will be used to examine the sensitivity of the thresholds that trigger each maintenance task. A wide range of different policies will be tested and applied to links and sections that are representative of the whole road network in Niger. The economic results will then be calculated. A series of iterations will enable national maintenance policies to be refined. These policies can then be applied to the link analysis to develop annual maintenance programs.

CONCLUSION

As was stated earlier, Niger is a country with very limited human and financial resources. It is therefore important that the best possible use be made of all investments, including highway maintenance. The Niger Road Maintenance Unit can ensure that funds are spent on those projects that give the highest returns by using the methods and techniques described in this study.

The views and interpretations expressed in this paper are those of the authors and should not be attributed to the World Bank, to its affiliated organizations, to the Government of Niger, or to any individual acting in their behalf.

Transportation Research Record 1106

Simplified Procedures to Manage the Maintenance of Low-Volume Roads

LOUIS BERGER AND JACOB GREENSTEIN

The maintenance requirements and work activities are considered for three types of low-volume roads: stone, natural gravel, and mechanically stabilized base course pavement (no blacktop). These road types provide about 90 percent of the transport accessibility in rural Ecuador. A description is provided of the activities that were performed during routine, periodic, and emergency maintenance operations. The required level of work for each activity is analyzed for each road type. The calculated work volume and known productivity of labor and equipment are used to determine the annual maintenance needs in terms of workdays of skilled and unskilled labor, and equipment type and quantity. In the next stage of analysis, a computerized unit price analysis is performed in conjunction with the annual maintenance budget. Once the cost analyses and annual maintenance budget are completed, the work schedule and financial distribution are planned. At this stage, a monthly or bimonthly detailed assignment for equipment and labor is performed to meet the maintenance needs for each activity; environmental constraints are also taken into account. The low-cost maintenance procedure and engineering properties of the materials used are described in detail.

Road maintenance plays an important role in minimizing transportation costs, increasing economic productivity, and improving the standard of living in the rural areas of Ecuador (*1*). Adequate maintenance is vital to maintain all-weather accessibility and minimize traffic hazards on unpaved low-volume roads. About 90 percent of the Ecuadoran low-volume rural roads that provide all-weather accessibility fall into the following three categories:

- Natural gravel roads are 4.0 to 6.0 m (13 to 20 ft) wide (see Figures 1 and 2, respectively). These roads are classified as Road Types 4 and 5, respectively (*1, 2*).
- Stone roads (empedrado in Spanish) are classified as Road Types 4E and 5E, and are 4.0 to 5.0 m (13 to 17 ft) wide, respectively. A typical stone road is shown in Figure 3.
- Mechanically stabilized base course pavements (no blacktop) are 6.0 m (20 ft) wide with 0.6 m (24 in) of shoulder on each side (see Figure 4). This road is classified as Road Type 6.

Natural gravel and stone roads are used for traffic levels of up to 100 vehicles per day (vpd) for the narrower road widths, and up to 150 vpd for wider road widths. The stabilized base course pavement is used for traffic volumes of 150 to 250 vpd. Experience with the performance and maintenance of these low-volume roads indicates that the most frequent causes of surface distress are as follows:

FIGURE 1 Typical 4.0 meters of gravel road.

FIGURE 2 Typical 6.0 meters of gravel road.

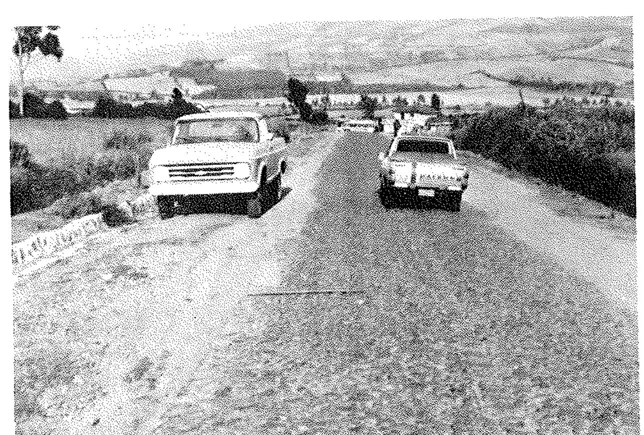

FIGURE 3 Mechanically stabilized base course pavement.

Louis Berger International, Inc., 100 Halsted St., East Orange, N.J. 07019.

FIGURE 4 Typical stone road.

- Dust formation, which is caused by traffic (wheel repetition), occurs mainly on the gravel roads and to a lesser extent on the base course roads.
- Raveling or disaggregation of the surface and loosening of the coarse aggregates occurs mainly on the gravel roads, less frequently on the base course roads, and least frequently on the stone roads.
- Rutting or plastic deformation occurs in all types of low-volume road surfaces, but with the greatest severity on gravel roads.
- Road surface erosion and potholes occur mainly in natural gravel roads, especially when the longitudinal slope is over 4 percent. Base course roads begin to deteriorate significantly when the longitudinal slope is over 7 percent. Stone roads rarely show erosion distress.
- Corrugation is typical in uncrushed gravel roads and is less common on base course roads.
- Roughness and upheaval distress occur in all road types because of subgrade movement or pavement instability.

Maintenance activities are defined as the specific work operations that are performed to minimize road distress or deterioration, and provide an adequate level of accessibility to the rural roads. The organization of the work into discrete activities enables the simplification and optimization of planning and the minimization of expenditures. The annual maintenance work is defined as the sum of the volume of work performed in all activities for the entire year. The annual volume of work for each activity is derived from the evaluation of road inventory data and the desired level of road service. Once the work needs and daily production for each activity are known, the annual and monthly requirements for equipment, labor, and materials can be determined.

The work programming is the final task of the maintenance planning stage. All maintenance activities are monitored and recorded to achieve the following objectives: (a) identify actual maintenance work done, (b) update equipment and labor production, (c) update and adjust road maintenance norms based on the road's performance record, and (d) update and adjust the equipment, labor, and maintenance needs for the rural road network.

ENGINEERING PROPERTIES OF RURAL ROADS

The all-weather rural roads in Ecuador are designed to provide safe traveling speeds of 25, 40, and 50 km/hr (15.25 to 32 mph) for mountainous, hilly, and level terrains, respectively (*1*). The design California bearing ratio (CBR) of the pavement material is 20 for Road Types 4 and 5, and 60 for Road Type 6. Pavements were designed according to the methods described in other studies (*3-5*).

The granular pavement thickness usually varies between 12 and 40 cm (5 and 16 in), depending on subgrade CBR, traffic loading, and environmental conditions. The maximum aggregate size is 5 to 7.5 cm (2 to 3 in) for Road Types 6 and 4 or 5, respectively. Pavement failure is defined as a rut depth of 7 to 10 cm (3 to 4 in) on top of the subgrade (*1, 4, 5*).

The stone road (or camino empedrado in Spanish) is constructed of two aggregate sizes: 10 to 15 cm (4 to 6 in) and 20 to 25 cm (8 to 10 in). The larger stone is defined as master stone and is used as an anchor, as shown in the typical cross-section depicted in Figure 5. The master stone is spaced at a distance of 1.0 to 1.5 m (3 to 5 ft) in the cross-section. It is used to develop the side pressure that increases the friction between the stones in the pavement, which in turn increases its bearing capacity.

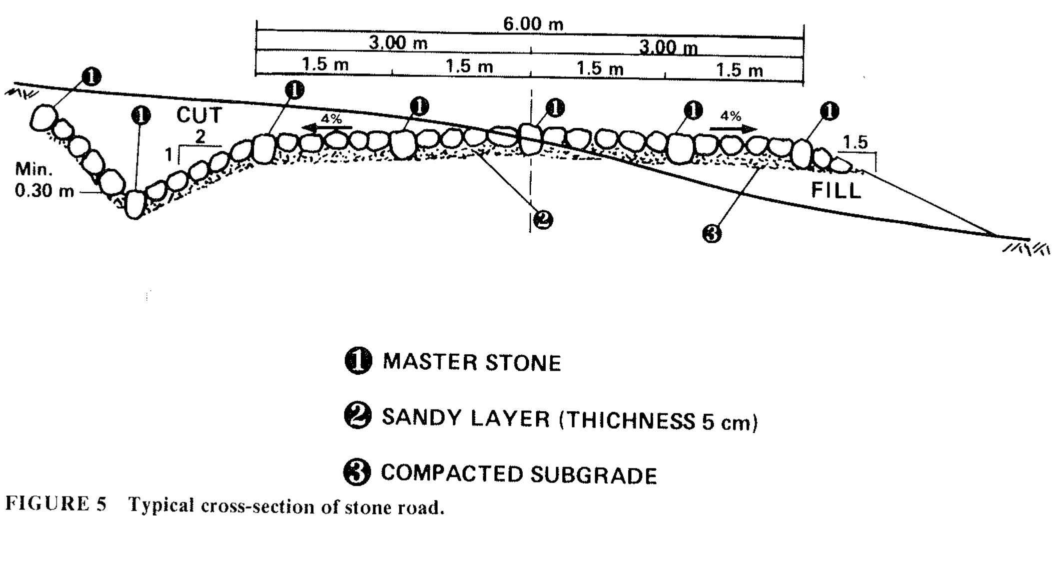

FIGURE 5 Typical cross-section of stone road.

The smaller rock (10 to 15 cm) is properly shaped and is located by skilled labor in between the master stone on top of a thin layer of nonplastic sandy material. The thickness of the sandy material is about 5 cm (2 in). New stone roads hardly require any pavement maintenance during the first 20 to 30 years. In other words, new stone roads give adequate accessibility with minimum maintenance costs during their entire economic lifetime, which is 17 years in Ecuador (*1, 6*).

MAINTENANCE ACTIVITIES

Each maintenance activity is performed to improve or eliminate a certain distress in each road element. For example, gravel and stone patching is performed to reduce surface erosion and potholes. Cleaning culverts and ditches, and controlling vegetation reduces surface rutting or plastic deformation. Regraveling or surface grading reduces raveling, surface disaggregation, and roughness. The definitions of the work activities needed to maintain Ecuadoran rural roads are given in Table 1 (*2*).

The activities defined in Table 1 are classified into three categories: routine (R), periodic (P), and emergency (E). These activities cover all maintenance work needed to maintain the pavement, drainage facilities, rights-of-way, bridges and water crossings, slopes, signs, and safety elements. Each maintenance activity in Table 1 is assigned a unique code. The average rate of daily production of each activity is also given. The variation of the daily production is attributed to the difference in climatic or topographical conditions and to the availability of local labor and materials.

The Ecuadoran climate plays an important role in the productivity of maintenance work. Three climatic zones can be distinguished: tropical, subtropical, and arid. The Ecuadoran tropical zones are located both in the Amazonas region east of the Andes Mountains and in a low-altitude narrow strip to the west of the Andes. The annual precipitation in these tropical zones is 2500 to 5000 mm/yr (100 to 200 in). The subtropical zone covers most of the Pacific Coast and the mountainous region, with an annual rainfall of approximately 600 to 2500 mm/yr (24 to 100 in). The arid zones have a precipitation of less than 600 mm/yr (24 in) and are located in the southern mountains and in some regions along the Pacific Ocean (Manabi).

The daily production for each activity shown in Table 1 was determined in 1985. For example, the daily production of gravel pothole patching varies mainly between 8 and 15 m^3/day. The lower value represents the average productivity in the higher altitudes, such as in the province of Chimborazo, and the higher value of production can be achieved in the lower altitudes or in the arid zone. Once the daily production for each maintenance activity is known, the daily requirements of labor, equipment, and materials can be determined by means of a maintenance work method statement (see Table 2).

A typical work statement is shown in this table for maintenance activity 111 R, gravel pothole patching. The work statements for all other activities can be found elsewhere (*2*). The maintenance statements are periodically updated to reflect current changes and adjust production rates.

It can be seen in the specific example of Table 2 that to produce 8 m^3 of gravel patching of graded aggregates, it is

TABLE 1 MAINTENANCE ACTIVITIES AND DAILY PRODUCTION

Code	Activity	Daily Production
111 R	Gravel pothole patching	8*-15 m³
112 R	Stone (empedrado) pothole patching	2*-3 m³
121 R	Cleaning of culverts	6-8* units
122 R	Ditch cleaning (labor)	1-3* ditch-km
131 R	Vegetation control (labor only)	0.5-0.6* ha
132 R	Small slide removal	8*-10 m³
133 R	Repair of small fill and edge failure	8*-10 m³
141 R	Cleaning of bridges and low-cost water crossings	2-4* units
211 P	Regraveling or pavement strengthening	250*-300 m³
212 P	Surface grading	3*-4 km
221 P	Ditch cleaning (Motor grader)	3-5* ditch/km
222 P	Replacement of culverts	1-2* units
231 P	Vegetation control (Machine)	1-2* ha
241 P	Bridge and ford maintenance and small repair	2*-3 units
251 P	Road sign servicing	6-8* units
310 P	Preparation of granular or gravel pavement material	100-200* m³
312 P	Preparation of stone pavement material (empedrado)	20-25* m³
313 P	Transportation of pavement materials	500-600* m³
314 P	Supervision and other periodic activity	20*-25 km
333 P	Other periodic maintenance	6*-10 work days
410 E	Machine slide removal	100-150* m³
411 E	Machine fill pavement reconstruction	60-80*-100 m³

R = routine, P = periodic, E = emergency.

* Represents the daily production of the province of Chimborazo.

TABLE 2 MAINTENANCE WORK METHOD STATEMENT

ACTIVITY: Gravel pothole patching   Code No. 111 R

Definition and Purpose: Manual repair of small surface area of gravel pavement with properly graded aggregate material to repair potholes, small depressions and other traffic hazard areas in order to restore a smooth riding surface.

Execution Criterion: Pothole deformation depth of 4 in. or when existing potholes are filled with water, creating traffic hazard.

Unit of Measure: m^3

Labor	Standard Work Method
1 foreman 8 unskilled laborers 1/3 driver	1. Loading and transporting aggregate materials 2. Placing safety devices and signs as required
Equipment 1/3 dump truck, (8-ton) 1 hand compactor 1 set of hand tools (4 shovels, 3 picks, 3 rakes)	3. Drying out excess water from potholes in tropical and subtropical zones 4. Placing select aggregate and smoothing out with rakes 5. Compacting with hand compactors and by moving trucks back and forth over patch
Materials Properly graded aggregate prepared and paid according to activity 310.	6. Leaving patch as smooth as possible 7. Removing safety devices and signs

Average Daily Accomplishment

$8m^3$

necessary to use one hand compactor, one-third of an 8-ton dump truck, and a complete set of hand tools. The required labor for this activity includes one foreman, eight unskilled laborers, and one-third of a driver. The maintenance work statement also gives the definition and purpose of each activity, the execution criterion, and a short description of the work method.

The required daily labor and equipment needed to achieve the daily production of work for each maintenance activity in the Ecuadoran mountainous province of Chimborazo are given in Tables 3 and 4, respectively. For example, to clean 5 km of ditches (activity 221 P, it is necessary to use one-third of a dump truck, a complete set of hand tools, and one motor grader.

In the case of emergency maintenance activity 410 E, one dump truck, one loader, one tractor, and one set of hand tools need to be operated by two operators, two assistants, one driver, two unskilled laborers, and one foreman to remove 150 m^3 of landslide material. The detailed labor requirement is shown in Table 3. Another example involves maintenance activities 111 R and 112 R, in which one foreman, eight unskilled laborers, and one-third of a driver are needed to produce 8 m^3 and 2 m^3 of gravel or stone pothole patching (see Table 1).

The additional daily requirements for a platform truck, a concrete mixer, water pumps, and mechanical saws are less frequently needed in the maintenance of rural roads.

ROAD AND BRIDGE INVENTORY

The principal objective of the inventory of rural roads and bridges is to provide updated information for the planning of maintenance needs in terms of the volume of work and the requirements for equipment and labor (*1, 2*). In other words, the inventory and road evaluation are used to optimize the maintenance expenditures for each road section in such a way that uniform accessibility will be provided to the users. Only a brief description of the four basic inventory elements is provided, as follows:

- *Identification of each road link*: Location, coding, and determination of the level of accessibility of the local population and agricultural production.
- *Engineering properties*: Length, width, type of road and terrain, number of culverts and bridges, type and length of ditches, and number of traffic signs.
- *Field evaluation*: Number and size of potholes, surface conditions, density and height of vegetation, slope failure hazards, performance of drainage facilities, types of soils, and material haulage distance.
- *Other observations*: Special maintenance needs and suggestions for repair of pavement, drainage facilities, bridges, fords, and other elements included in the right-of-way.

One engineering team can inventory and evaluate approximately 50 km of rural roads a day.

MAINTENANCE WORK PLANNING

Annual maintenance work plans describe which operations should be performed, in quantity and quality, to meet a predetermined user service level. The selected service level will establish costs for both the maintenance effort and the users. In order to keep vehicle operating costs (VOCs) at an absolute minimum, the rural road network would have to be maintained

TABLE 3 DAILY LABOR REQUIREMENTS

Activity	Foreman	Unskilled Laborers	Drivers	Motor Grader Op.	Roller Op.	Op. Assistance	Mason	Mower Op.	Loader Op.	Tractor Op.	Engineer
111 R	1	8	.33								
112 R	1	8	.33								
121 R	1	8	.33								
122 R	1	8	.33								
131 R	1	8	.33								
132 R	1	8	.33								
133 R	1	8	.33								
141 R	1	8	.33								
211 P	1	2	1.33	1	1	2					
212 P	1	2	1.33	1	1	2					
221 P	1	4	.33	1	1	1					
222 P	1	8	.33				1				
231 P	1	4	.33					1			
241 P	1	8	.33								
251 P	1	4	.33								
310 P	1	2	2			2			1	1	
312 P	1	8	.33								
313 P			1								
314 P			1								1
333 P	1	4	.33								
410 E	1	2	1			2			1	1	
411 E	1	2	1			2			1	1	

TABLE 4 PRINCIPAL DAILY MAINTENANCE EQUIPMENT REQUIREMENTS

Activity	Dump Truck (8-ton)	Hand Roller	Hand Tools	2.5-8.0 Ton Vibratory Roller	Water Tanker	Motor Grader	Mower	Loader	Tractor	Screening Plant	Pickup (2.5-ton)
111 R	.33	1	1								
112 R	.33		1								
121 R	.33		1								
122 R	.33		1								
131 R	.33		1								
132 R	.33		1								
133 R	.33	1	1								
141 R	.33		1								
211 P	.33		1	1	1	1					
212 P	.33		1	1	.5	1					
221 P	.33		1			1					
222 P	.33		1								
231 P	.33		1				1				
241 P	.33		1								
251 P	.33		1								
310 P	2							1	1	1	
312 P	.33		1								
313 P	1										
314 P											1
333 P	.33		1								
410 E	1		1					1	1		
411 E	1		1					1	1		

to a nearly as-built condition. This, however, would not be physically possible without first making a very large expenditure to regravel and reconstruct the granular pavement surface. Furthermore, even if this could be accomplished, it is not usually economically prudent to maintain a rural road system at the as-built top-service level. The reason for this is that minimizing user costs would maximize maintenance costs, as shown in Figure 6.

The relationship between user costs and maintenance effort is real and is exponential in nature. An economic trade-off exists between user and maintenance costs. The optimum, at which total costs are at a minimum (see Figure 6), is the desirable level of service. This level of service varies with traffic volume, topography, climatic conditions, and type of road surface. The optimum level of maintenance service for a mountainous rural region is presented in Table 5 (*1, 2, 6*). For example, the maintenance service level of activity 111 R, gravel pothole patching, is 4.0 and 6.0 m^3/km road for Road Types 4 and 5, respectively. This service level was determined as follows (*2*).

In order to achieve the optimum level of maintenance service in Chimborazo, it was found that 1/100 of the road's surface should be patched annually with an average patching depth of

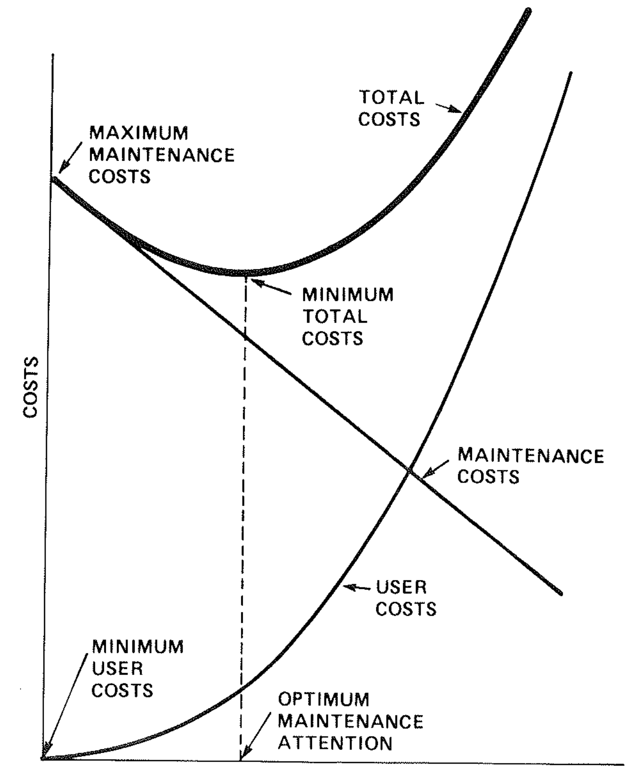

FIGURE 6 Maintenance costs versus user costs.

10 cm. The pavement width of Road Types 4 and 5 is 4.0 and 6.0 m, respectively. Therefore, the service level of gravel patching is 1000 m × 4.0 m × 0.1 m × (1/100) = 4 m^3/km for Road Type 4, and 6 m^3/km for Road Type 5. The optimum level of service was determined in a similar manner for other maintenance activities, and the results are shown in Table 5.

Once the optimum level of service is established and the road inventory is completed, the annual maintenance needs can be calculated for each activity. For example, the annual needs for gravel pothole patching are calculated as follows:

$$GPP_{ijk} = \sum_{i=1}^{i} \sum_{j=1}^{j} \sum_{k=1}^{k} LNG_{ijk} \cdot SL_{ijk} \cdot AF_{ijk}$$

where

GPP = the total annual gravel pothole patching in m^3 of activity 111 R for all road types (i=4, j=5, and k=6);
LNG = the lengths of all road types (i=4, j=5, and k=6);
SL = the service level of gravel patching in m^3/km given in Table 5 for Road Types 4, 5, and 6; and
AF = the adjustment factor of the current road condition, which is determined from the road inventory and evaluation (*2*).

For example, the surface of 3.3 km of Road Type 5 was evaluated in the inventory as being in very poor condition. The engineering interpretation of such surface condition is that the volume of maintenance work should be increased by 20 percent to provide optimum road accessibility (*2*). In other words, the adjustment factor, AF, equals 1.2. According to Table 5, the annual service level (SL) of patching is 6.0 m^3. Therefore, the annual gravel patching needs of activity 111 R for this road are 3.3 km × 6.0 m^3/km × 1.2 = 23.8 m^3/year.

In addition to the work needs that were determined by the overall maintenance plan just described, other maintenance needs result from the following conditions:

- Deficiencies that were encountered during regular field inspection,
- Deficiencies that were noticed by crew foremen while performing another work activity, and
- Requests for maintenance services by users, usually in the form of complaints and emergencies.

TABLE 5 ANNUAL MAINTENANCE LEVEL OF SERVICE

Activity	Unit of Measurement	Level of Service by Road Type				
		4	5	4E	5E	6
111 R	m^3/km of road	4.0	6.0	-	-	6.0
112 R	m^3/km of road	-	-	4.0	6.0	-
121 R	Percentage of all culverts	50%	50%	50%	50%	50%
122 R	Percentage of existing ditches	50%	50%	50%	50%	50%
131 R	ha/km of road	0.1	0.1	0.1	0.1	0.1
132 R	m^3/km of road	2.0	2.0	2.0	2.0	2.0
133 R	m^3 of road	0.5	0.5	0.5	0.5	0.5
141 R	Percentage	100	100	100	100	100
211 P	m^3/km of road	60	90	-	-	90
212 P	Percentage of road surface	100	100	-	-	200
221 P	Percentage	50	50	50	50	50
222 P		20	20	20	20	20
231 P	ha/km of road	10	10	10	10	10
241 P	Percentage	100	100	100	100	100
251 P	Percentage	50	50	50	50	50
310 P	m^3/km of road	64	96	-	-	96
312 P	m^3/km of road	-	-	4.0	6.0	-
313 P	m^3km/km of road	1,600	2,400	100	150	2,400
314 P	km/km of rod	2	2	2	2	2
333 P	Man-days/km of road	0.5	0.5	0.5	0.5	1.0
410 E	m^3/km of road	10	10	10	10	10
411 E	m^3/km of road	2	2	2	2	2

Requests for services by users of unpaved roads should not be treated lightly. As a matter of fact, public awareness of maintenance needs should be encouraged to reduce total efforts and costs. For example, if a foreman reports a buildup of debris at the entrance to a culvert and the debris is cleared before the next storm, road failure and traffic hazards can be avoided. It is recognized that although annual work planning programs define theoretical overall work objectives, short-term schedules must be responsive to actual needs. Therefore, deviations from work programs may be required and should be expected. An example of the annual maintenance needs for 1000 km of rural roads in Chimborazo is given in Table 6. This inventory includes the following road types: (a) 350 and 200 km of gravel Road Types 4 and 5, respectively; (b) 150 km of stone Road Type 4E and 150 km of Road Type 5E; and (c) 150 km of crushed base Road Type 6.

Along the 1000 km of road are 20 bridges and about 330 culverts, or about one culvert every 3 km. There are also 500 km of ditches, two-thirds of which are cleaned by labor; one-third are cleaned by machines. There are also 100 signs along these roads.

The volume of maintenance work for each activity and each road type is shown in Table 6. These work quantities were calculated by multiplying the road's length by the annual maintenance level of service (given in Table 5). For example, as shown in Table 5, the annual level of service is 4.0 and 6.0 m^3/km of activity 111 R for Road Types 4 and 5, respectively. The annual maintenance need is therefore 350 km $\times$ 4 m^3/km = 1400 m^3 and 200 km $\times$ 6 m^3/km = 1200 m^3, respectively (see Table 6). The annual work for all Road Types 6 is 150 km $\times$ 6 m^3/km = 900 m^3. The total annual needs for activity 111 R are 1400 + 1200 + 900 = 3500 m^3. The annual needs for stone patching (activity 112 R) for 300 km of stone roads are 1500 m^3. The annual needs in terms of work volume for each activity and for the total maintenance work shown in Table 6 are used to program the annual requirements for labor and equipment.

Annual Programming for Labor and Equipment

Annual maintenance labor and equipment programming is based on (a) the total annual work volume for each activity, as shown in Table 6, (b) the daily production of each activity, as shown in Table 1, and (c) the requirements for labor and equipment needed to meet this daily production, as shown in Tables 3 and 4, respectively.

For example, the annual work volume of activity 111 R (gravel pothole patching) is 3500 m^3 (Table 6). The daily production is 8 m^3 (Table 1). Therefore, the annual crew day is 3500/8 = 437 crew days. As shown in Table 3, each crew day includes one foreman, eight unskilled laborers, and 0.33 drivers. Therefore, the annual number of workdays for activity 111 R is 437 $\times$ 1 = 437 workdays for the foreman, 437 $\times$ 8 = 3496 workdays for unskilled laborers, and 437 $\times$ 0.33 = 144 workdays for drivers. The sum of all required workdays for all 22 maintenance activities is the annual programming for labor. The total required workdays needed to maintain 1000 km of rural roads in Chimborazo is shown in Table 7.

The annual programming of equipment days (EDs) is performed in a similar manner. The total annual number of EDs needed to perform activity 111 R is 3500/8 = 437 EDs. As shown in Table 4, each ED includes 0.33 dump trucks, one manual compactor, and a complete set of hand tools for eight unskilled laborers. Therefore, the annual number of EDs needed to perform activity 111 R is 437 $\times$ 0.33 = 145 EDs for the dump truck, 437 $\times$ 1 = 437 EDs for the manual roller, and 437 $\times$ 1 = 437 EDs for hand tools.

TABLE 6 RELATIONSHIP BETWEEN ANNUAL MAINTENANCE NEEDS AND ROAD TYPE (1000 KM OF RURAL ROADS)

Activity	Road Length	4	4E	5	5E	6	Total Annual Work Volume	Unit Price (sucres)*	Total Annual Cost (sucres)
		350	150	200	150	150			
111 R	m^3	1400	0	1200	0	900	3500	1817	6,359,500
112 R	m^3	0	600	0	900	0	1500	6228	9,342,000
121 R	U	525	225	300	225	225	1500	2055	3,082,500
122 R	km	58	25	33	25	26	167	5409	903,303
131 R	ha	35	15	20	5	20	105	20553	2,158,065
132 R	m^3	700	300	400	300	300	2000	1713	3,426,000
133 R	m^3	175	75	100	75	75	500	2303	1,151,500
141 R	U.	2	3	3	4	8	20	4111	82,220
211 P	m^3	21000	0	18000	0	13500	52500	326	17,115,000
212 P	km	350	0	200	0	200	750	23645	17,733,750
221 P	km	30	12	17	12	12	83	7860	652,380
222 P	U	210	90	120	90	90	600	7672	4,603,200
231 P	ha	2.5	2.5	3	3	4	15	7715	115,725
241 P	U	2	3	3	4	8	20	8473	169,460
251 P	U	35	15	20	15	15	100	1515	151,500
310 P	m^3	22400	0	19200	0	14400	56000	478	26,768,000
312 P	m^3	0	600	0	900	0	1500	685	1,027,500
313 P	m^3-km	560000	15000	480000	22500	360000	1437500	16	23,000,000
314 P	km	700	300	400	300	300	2000	310	620,000
333 P	h/day	175	75	100	75	150	575	1650	948,750
410 E	m^3	3500	1500	2000	1500	1500	10000	554	5,540,000
411 E	m^3	700	300	400	300	300	2000	1351	2,702,000

Total: Sucres 127,652,353
U.S. $ 1,329,712

*U.S. $1 = 96 sucres

TABLE 7 TOTAL REQUIRED ANNUAL WORKDAYS TO MAINTAIN 1000 KM OF RURAL ROADS

Definition of Labor	Foreman	Unskilled Laborers	Drivers	Motor Grader Operator	Roller Operator	Operator Assistants	Mason	Mower Operator	Loader Operator	Tractor Operator	Engineer
Total Annual Work Days	3200	20,000	4000	500	480	1700	300	400	150	400	100

TABLE 8 TOTAL REQUIRED ANNUAL EQUIPMENT DAYS TO MAINTAIN 1000 KM OF RURAL ROADS

Equipment Type	Dump Truck (8-ton)	Hand Roller	Hand Tools	2.5-8.0 Ton Vibratory Roller	Water Tanker	Motor Grader	Mower	Loader	Tractor	Screening Plant	Pickup (2.5-ton)
Total Annual Equipment Days.	4000	500	3000	450	300	500	150	400	400	300	100

TABLE 9 LABOR REQUIRED TO MAINTAIN 1000 KM OF RURAL ROADS

Professional Class	Number of Professionals
Foreman	16
Unskilled labor	100
Drivers	22
Motor grader operator	4
Roller operator	3
Loader operator	3
Tractor operator	3
Mower operator	1
Assistant operator	9
Mason	2
Engineer/supervisor	1

TABLE 10 ANNUAL EQUIPMENT REQUIRED TO MAINTAIN 1000 KM OF RURAL ROADS

Type of Equipment	Amount
Tractor (D7)	1
Tractor (D6)	2
Water tank (2,000 gal)	2
Dump truck	20
Motor grader	4
Vibratory roller	3
Hand roller	3
Loader	3
Mechanical mower	1
Portable water pump	2
Small concrete mixer	1
Screening plant	2
Pick-up truck	1
Communication units	3
Portable generator	1
Platform (25-ton)	1
Maintenance truck	1
Mechanical saw	3
Small water tank (200 gal)	1

Another example in regard to regraveling (activity 211 P) is given in Table 6, in which the total annual volume of work is 52,500 m^3/1000 km of road. The daily production of a crew day (CD) is 250 m^3 (see Table 1), therefore the annual CD or ED is 52,500/250 = 210 CDs or EDs. As shown in Table 3, each ED includes 0.33 dump trucks, one vibrator roller, one water tank, and one motor grader. The annual number of EDs is therefore $0.33 \times 210 = 70$ EDs for the dump truck and 210 EDs each for rollers, water tank, and motor grader. It should be noted that the preparation of material and its transportation to the road is included in other activities, namely 310 and 313, respectively. The total required EDs for the principal maintenance equipment that is needed to maintain 1000 km is given in Table 8.

The quantity of labor and equipment is calculated in the final stage of programming. The annual number of working days in the field should be determined to complete this stage. This number is a function of the maintenance needs, weather conditions, and type of activity. The foremen, drivers, and unskilled laborers are working approximately 200 days/yr in the field and the operators are working on the road about 150 days/yr. Loaders and screening plants are being used 100 days/yr. In addition to the equipment shown in Table 8, a rural road maintenance unit also needs a platform, a maintenance truck (25-ton), communication units, mechanical saws, generators, a concrete mixer, and portable water pumps. The typical labor and equipment that is needed to maintain 1000 km of rural roads is shown in Tables 9 and 10.

Scheduling Procedures

Detailed monthly scheduling enables the field crew to clearly understand its short-term maintenance work objectives. Experience indicates that, without a sound, short-term schedule, crews frequently perform the easiest and most convenient tasks instead of those that are actually required. Experience in Ecuador indicates that work schedules for rural roads should be prepared for monthly periods. The actual preparation should be undertaken as a joint effort between the engineers and foremen who will be responsible for the work. Once the requirements and needs for labor, equipment, and materials have been determined and before the schedules are prepared, consideration must be given to the following:

- Timing (when the work must be performed),
- Location (where the work must be performed),
- Local obstacles (which mainly result from bad weather conditions),
- Duration (how long the work will take), and
- Resources (requirements for equipment, personnel, and materials).

Typical monthly work schedules in Chimborazo indicate that 5 and 10 percent of the annual resources for gravel and stone patching (activities 111 R and 112 R) are performed monthly from January to April and from May to December, respectively. In other words, the expenditure of surface patching can be made better during the relatively dry season from April to December. Another schedule indicates that equal amounts of bridge and ford maintenance work are performed from March to December. No such activity is undertaken in the stormy season during January and February. Culverts can be cleaned properly only in the dry season between June and December.

Preparation of Annual Budgets

The preparation of the annual budget is the last task in the maintenance management procedure. This task is rather simple once the annual and monthly resource needs for equipment, labor, and materials for each work activity are known. A detailed unit price analysis is performed to achieve an accurate cost estimate. A simple, computerized cost analysis that includes the economic costs of equipment, labor, material, haul distance, and rate of production is completed to determine the direct cost for each maintenance activity (*1, 2*). For each direct cost there are other indirect expenditures, such as overhead, contingencies, supervision, and profit if the work is performed by a private contractor. The representative unit cost for each work activity in local currency ($1 (U.S.) = 96 sucres in 1985) is given in Table 6. For example, the unit price of gravel and stone patching activities 111 R and 112 R is 1817 and 6228 sucres/m^3, respectively. The total annual budget needed in 1985 to maintain 1000 km of rural roads was 127,652,353 sucres or $1,329,712, which is about 4 to 5 percent of the road construction cost.

Report Monitoring and Work Control

The maintenance management of rural roads only requires the use of a simple field report that can easily be implemented on the site by the foreman or a senior crew member to report and control work. The purposes of the field report are as follows:

- To identify and control the volume of work for each maintenance activity for each road,
- To update and adjust the rate of production for the different activities and road locations, and
- To update the maintenance needs and requirements for labor, equipment, and materials.

The field report includes accurate and reliable descriptions of (a) work location; (b) work accomplished; (c) personnel, by name, labor classification, and number of hours/week; (d) equipment, by type, identification number, and hours assigned to the activity; (e) materials; and (f) comments on any special problem observed during the actual field operation. This information can easily be stored in a personal computer and has been found to be very useful in increasing production and improving the quality of maintenance work. In addition, the field information is valuable for any statistical analysis regarding the conditions and needs of maintenance equipment.

SUMMARY AND CONCLUSIONS

A simple methodology was presented for planning the maintenance of low-cost gravel, stone, and base course roads without blacktop. The pavement width of these roads varies between 4.0 and 7.2 m (13 to 24 ft) and the roads are designed to carry up to 250 vpd. The different rural roads used in Ecuador are shown in Figures 1 to 4. The engineering classification of these roads is also presented.

The 22 work activities for routine, periodic, and emergency maintenance are shown in Table 1 together with daily work production. The methodology for determining the labor, equipment, and materials required for each maintenance activity is presented and a typical example related to gravel potholes demonstrates the method of calculation (Table 2).

The optimum maintenance level is defined as the work effort needed to minimize the total expenditures of both road users and maintenance costs. The optimum maintenance level of service varies with traffic volume, topography, climatic conditions, and type of road surface. A typical optimum level of maintenance service for the rural Ecuadoran mountainous region is presented in Table 5. When these levels of service were implemented, the rural roads were maintained in good condition and the VOC was approximately $0.3 to $0.5/km. When the level of maintenance was reduced, the VOC increased from $0.5 to approximately $0.8/km. The upper-limit VOC is related to zero maintenance and a surface roughness of over 10 m/km.

The annual maintenance needs for each activity are determined from a simplified road and bridge inventory and evaluation, the criterion of optimum level of maintenance service, deficiencies that were encountered during regular field inspection, deficiencies that were noticed by crew foremen while performing other work activities, and requests for services by users.

Once the total annual maintenance needs are known the requirements for equipment and labor can be determined in terms of equipment days and workdays. A total of 10,000 equipment days and 13,300 workdays are needed to maintain 1000 km of rural roads in the mountainous region of Ecuador. For local conditions, this requires about 16 foremen, 100 laborers, 22 drivers, 23 different equipment operators, and their assistants under the overall supervision of a road maintenance engineer. The following basic heavy equipment is used to maintain these roads: four motor graders, three tractors, twenty dump trucks, two water tankers, six vibratory and hand rollers, three loaders, two screening plants, one maintenance truck, one platform, and other smaller pieces of equipment (Table 10). An optimum cost level of maintenance service is obtained when the annual expenditures are approximately 4 to 5 percent of the initial economic construction costs.

A monthly work schedule is needed to ensure an adequate maintenance operation. This work schedule involves timing and location; local obstacles, which mainly result from water conditions; duration of the operation; and the resources required for equipment, personnel, and materials.

A simple procedure is needed to control the quality and quantity of work, update and adjust the rate of production, and update maintenance needs and requirements for labor, equipment, and materials.

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Road Maintenance Costs and Research Directions of Low-Volume Logging Roads in New Zealand

D. M. ROBINSON AND P. J. FARLEY

Road maintenance costs are a significant consideration in the management of pavements for logging traffic. Cost and volume data have been gathered from a number of logging roads, within a common forest area, over a 15-yr period. Axle loads were usually close to the legal highway limits (8.2 tonnes equivalent single-axle) and volumes ranged from 2,000 to 250,000 tonnes/yr. A regression analysis of the data confirmed that maintenance costs on unsealed gravel pavements increase as the volume of logs carted over the road increases. A sealed road that was constructed to a standard that gives low Benkelman beam deflections showed an expected decrease in maintenance cost; this relationship is quantified. Overall, the data showed a wide range of maintenance costs versus volume hauled, with no strong statistical correlation between the two. The observations presented can be readily upgraded to allow for inflation (using a construction cost index). As more data become available, they can be used as one component in a multivariable model to optimize the design, rehabilitation, or reconstruction of a road, and improve the economics of a log transportation system. Even without this sophisticated modeling, they can provide valuable indicators for roading economies. Research directions within the New Zealand logging industry are detailed with particular emphasis on the means by which the industry can be effectively informed of known techniques in planning, economics, and construction. General comments are made on current and expected research and extension work.

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The New Zealand forest industry is becoming increasingly aware of rising costs as harvesting moves into plantation forests that were established on steep and difficult terrain. "Old crop" forests that were established on easy terrain during the early 1930s as part of the government's depression employment scheme have mostly been harvested. The "new crop" forests that were planted since 1950 are now coming into production. It is becoming apparent that the economic justifications used in the establishment of these forests were excessively optimistic, particularly in the state sector in which soil stabilization and employment opportunities were viewed as additional objectives. The requirement to provide an economic return on investment was not given primary importance.

New Zealand's economic philosophy is undergoing a major change, from a production base that is directly or indirectly subsidized to one that is unsubsidized and market-driven. The emphasis has been diverted from attempts by the government to assist industries or sectors that are seen as desirable, or successful, toward a situation of government neutrality. This change has led to alterations in the tax structure that now give less relief to farming and forestry industries. The cost of all aspects of plantation forestry, particularly timber harvesting, is being closely examined.

Because roading and transportation may account for up to 50 percent of this harvesting cost, an appreciation of its components and their interrelationships is essential. Models for predicting cost, even if simple, are of importance in identifying an economic return, or at least, in minimizing the cost of logging transportation.

The present network of forest roads in New Zealand is approximately 26,000 km in length and a 50 percent increase is expected over the next 15 years as timber harvesting expands. The road types are as varied as the topography. Approximately 50 percent of the plantation forests are state-owned and the remainder are in private ownership. Of the major forest roads, the following two types are the most important:

- Gravel or aggregate pavement with no asphaltic or bitumen seal coat, and
- Chip-sealed pavement that consists of a compacted aggregate basecourse finished with a layer of competent, crushed aggregate (chip) held by a minimal application of bitumen.
  - The typical chip size is 7.5 to 12.0 mm, average least dimension, applied at rates of 60 to 80 m^2/m^3, and
  - The typical bitumen application rates are 1.3 to 1.7 liters/m^2.

Much of the current forest roading practice, both in terms of design and maintenance, has followed that of the national roading network. Forest roading, however, differs in emphasis from that of the national network, notably in regard to heavy vehicle distribution, traffic volumes, and available, or affordable, maintenance resources. Many traditional practices are now increasingly subject to investigation and change, including aggregate grading envelopes and specifications, resealing frequency, partial road width or wheel track only resealing.

Much of the roading in the forest industry is performed by personnel with little or no formal training in engineering or roading. This, and the realization that some practices have not been particularly cost-effective, means that more extensive planning and investigation of all aspects of forest roading is needed. Cost justification, cost-benefit analysis, and the economic realities of a market-oriented economy (i.e., profit or perish) put further pressure on traditional practices.

TRANSPORT COSTS

The following are the three main components of transport cost:

- *Road construction costs.* The traditional engineer's estimate or a variation of construction costs is well-understood and likely to be backed up by historical cost records from either contract or in-house construction (*1*).
- *Road maintenance costs.* The true, historical background costs are often unreliable or hidden for a number of reasons that may include tax avoidance, capital cost charged to maintenance; area costing instead of road-by-road costing.
- *Transport or trucking costs.* The relationship of truck owning and operating costs to the total harvesting cost is well-understood. Much less understood in any readily quantifiable form are the interrelationships between the following:
  - Road roughness and operating costs;
  - Geometric standards, gradients, and travel time;
  - Operating costs and road type (i.e., increased brake, tire, and general maintenance caused by unsealed roads).

As will be described later, the National Roads Board (NRB) and the Logging Industry Research Association (LIRA) of New Zealand are performing trials in some of these cost categories. Recent work by the NRB is useful in relating road roughness to vehicle operating costs (*2*).

Whether an analysis of logging transport systems is performed through network analysis or by use of conventional cost-benefit analyses of road improvements or upgrading, the single, most uncertain item is maintenance cost (*3*). Although it is not the major cost in harvesting, the fact that it is not adequately quantified is a hindrance to a realistic assessment of its position during a rational analysis.

ROAD MAINTENANCE COSTS

In an attempt to find reliable historical costs for road maintenance, the following items had to be considered, because historical road maintenance costs often included capital construction costs for various reasons.

- Uncertainty exists as to what is maintenance and what is capital expenditure.
- Taxation benefits may be greater from maintenance than from capital expenditure, and it is often difficult to apportion them accurately at a later stage.
- Road maintenance is often performed on a wet day as a substitute for regular work, and is performed regardless of its need.
- In small forests, road maintenance is often used as a justification for purchasing or keeping plant, such as graders, bulldozers, and trucks; this also leads to maintenance being performed regardless of its need.

When considering these points, the most reliable information, in fact the only information that is readily available on a road-by-road basis, is that used in this analysis.

In May 1971, the New Zealand Forest Service Head Office required the establishment of a record of maintenance costs and volume of timber carted on the main roads in state forests. Useful data were accumulated in the only place in which this was done by adding the road number to the accounting code for all maintenance expenses.

Timber Volumes

All harvested timber volumes are recorded in detail by compartments. The total volume extracted from each compartment was recorded against the roads that were used to transport the timber in the particular time period. In this case, the task of assigning compartment volume flow to a particular road was straightforward, because in almost all cases only a single road link was available.

In addition to forest timber, a substantial volume of other timber has been carted over some of the roads used in this analysis in recent years. This additional traffic used these roads in preference to the state highway system to minimize the cartage distance and national road user charges. These operators pay a fixed fee per load for the use of the forest roads. The volume of timber that was generated outside the forest and transported on these roads has been reliably estimated from the record of payment. This fixed-fee system is unrealistic in accounting for the cost of road maintenance, but is easy from an administrative standpoint and does not lead to substantial losses. Other traffic that comprises agricultural and general transportation, and overweight vehicles that were diverted from an inadequate state highway, has not been fully recorded. This traffic, which mainly travels over Roads 1 and 20, is not likely to significantly affect the results of this study. No account was taken of conventional light traffic.

Road Details

ROAD 1, Valley Road, is a 13.8-km, two-lane road situated on well-drained alluvial river flats adjacent to the Motueka River. It had been used for many years before an overlay was applied and it was resealed in March 1972. Benkelman beam deflections that were taken after it was resealed ranged from .3 to .6 mm.

ROAD 20, Stock Road, is a 7.2-km, two-lane road situated on weathered glacial outwash gravels that cross a low saddle between the Motueka and Wai-iti Rivers. It follows the general line of an old road but was constructed on a new alignment from 1973 to 1975. Approximately 40 percent of the road was sealed in March 1973; the balance was sealed in March 1975. Benkelman beam deflections that were taken after it was sealed ranged from 1.0 to 1.5 mm. Some maintenance problems associated with poor subsurface drainage have been evident since shortly after construction.

ROAD 2, Kerrs Hill Road, is an 11.4-km, established, one-and-a-half-lane road situated on weathered glacial outwash gravels that cross a ridge between the Motueka and Motupiko Rivers. In 1978, 4.5 km of the road was realigned and upgraded to two lanes; the balance was realigned and upgraded in 1980. The road was partially sealed in 1982 and was completed by March 1983.

ROAD 8, Blows Road, is a 4.0-km, one-and-a-half-lane road situated on a mixture of alluvial and glacial outwash gravels. It runs adjacent to a tributary stream of the Motueka River and provides access for logging that catchment area.

Cost Data

Typical cost data for Road 1 are summarized in Table 1 and Figure 2. All costs have been adjusted using the Ministry of Works and Development Construction Cost Index (CCI) to allow for inflation. A standard computer spreadsheet package was used to allow the cost information to be easily updated using the CCI, which accounts for cost increases in labor, plant, and materials. Maintenance costs are plotted against volume for all four roads in Figures 1 to 4. The costs and volumes are for 6-month periods. The unit cost (cost/volume) is not time-dependent and can be used for direct comparison with other costs. An allowance has been made for the fact that some of the costs on any given road are for both aggregate and sealed surfacing and for sealing costs charged to maintenance (i.e., Road 1: 1971 to 1972; 1985).

RESULTS OF THE ANALYSIS

Sealed Roads

Analysis of Road 1 shows no good linear correlation, either single or multiple, between cost and volume or time. The unit cost (cost/volume) shows no good correlation with time. However, the unit costs that are plotted against time since sealing in Figure 5 provide some indications that may be of practical value.

No statistically significant correlation exists in Road 20 between cost and volume or time (Figure 2), and unit cost and time since sealing (Figure 5). However, the plot of unit cost against time since sealing (Figure 5) gives some practical indication of the expected range of maintenance costs.

Unsealed Roads

The simple linear correlation between cost and volume is not good (R^2 = 12 to 16 percent), as shown in Figures 3 and 4. When costs and volumes were smoothed over three periods better correlations were obtained for Roads 2 and 8, as shown.

$$\text{Cost of Road 2} = \$NZ537 + \$NZ0.020 \times \text{volume (tonnes)} \quad (R^2 = 51.2 \text{ percent})$$

$$\text{Cost of Road 8} = \$NZ466 + \$NZ0.038 \times \text{volume (tonnes)} \quad (R^2 = 28.5 \text{ percent})$$

Although the second equation does not have great statistical weight, these results provide an indication of road maintenance costs in the absence of anything better.

COMMENTS

The data for the sealed roads show considerable scatter. This is to be expected because sealed road maintenance in particular is characterized by two features. First, the need for maintenance often can only be seen months after the concentration of loads has passed. Second, sealed road maintenance work is generally not urgent. It is also usually performed by off-site contractors, which results in a further delay between damage and repair.

The two roads do not show the same cost relationships. This may be accounted for in the differing initial construction standard and the differing maintenance requirement that results from topography and generally poor drainage of Road 20.

The initial Benkelman beam deflections that were taken on Road 1 were considerably less than those on Road 20. The indications are that this has resulted in lower maintenance costs and less sensitivity to the volume of heavy traffic.

Considerable data scatter is inevitable on unsealed roads as a result of the following factors:

- The influence of the time of the year at which loads are transported (also true for sealed roads);
- The specific weather conditions during periods of particularly heavy logging (also true for sealed roads);

TABLE 1 TYPICAL ROAD MAINTENANCE COSTS AND TIMBER VOLUMES

ROAD NO. 1

DATE	C.C.I.	VOLUME (000 tonnes)	COST $NZ/Km	ADJ COST $NZ/Km	REMARKS
July-Dec 1971	981	12	1723	4277	Includes some sealing cost
Jan-Jun 1972	981	65	1631	4048	Includes some sealing cost
Jul-Dec 1972	981	82	143	355	
Jan-Jun 1973	981	114	2	5	
Jul-Dec 1973	981	79	2	5	
Jan-Jun 1974	981	91	92	228	
Jul-Dec 1974	981	120	163	405	
Jan-Jun 1975	981	104	163	405	
Jul-Dec 1975	981	99	71	176	
Jan-Jun 1976	981	51	51	127	
Jul-Dec 1976	981	21	377	936	
Jan-Jun 1977	981	38	367	911	
Jul-Dec 1977	981	91	31	77	
Jan-Jun 1978	981	75	102	253	
Jul-Dec 1978	981	81	143	355	
Jan-Jun 1979	1027	55	39	92	
Jul-Dec 1979	1119	47	288	627	
Jan-Jun 1980	1318	32	178	329	
Jul-Dec 1980	1450	34	154	259	
Jan-Jun 1981	1590	35	29	44	
Jul-Dec 1981	1750	37	97	135	
Jan-Jun 1982	1870	37	204	266	
Jul-Dec 1982	2000	30	500	609	
Jan-Jun 1983	2030	52	75	90	
Jul-Dec 1983	2030	54	155	186	
Jan-Jun 1984	2060	32	50	59	
Jul-Dec 1984	2195	28	213	236	
Jan-Jun 1985	2365	45	1363	1403	Includes some reconstruction cost
Jul-Dec 1985	2435	42	584	584	

- The influence of the construction standard on subsequent maintenance (also true for sealed roads); and
- The fact that periodic cumulative maintenance, especially the replacement of top course aggregate, does not necessarily coincide with periods of heavy loading.

TRENDS

Data are sufficient to indicate the existence of (a) a weak positive relationship between tonnes of timber carted and maintenance costs for unsealed roads, (b) a very definite divergence in the costs for sealed and aggregate roads, and (c) the general cost level of sealed road maintenance.

Relationship Between Tonnes of Timber and Equivalent Design Axles

An average logging truck carries about 20 to 25 tonnes of payload and is assessed as being equivalent to 2.5 equivalent design axles (EDAs). One-thousand tonnes of timber is therefore equivalent to 100 to 125 EDAs. (1 EDA = 8.2 tonnes on a twin-tired single axle.)

This assessment compares with the NRB standard assessment method using commodity factors: 1.35 EDAs per logging truck. However, this method counts both full and empty trucks. Therefore, EDAs/truck are equal to 2.7 (allowing for two-way travel). There are therefore approximately 135 EDAs/1000 tonnes of payload.

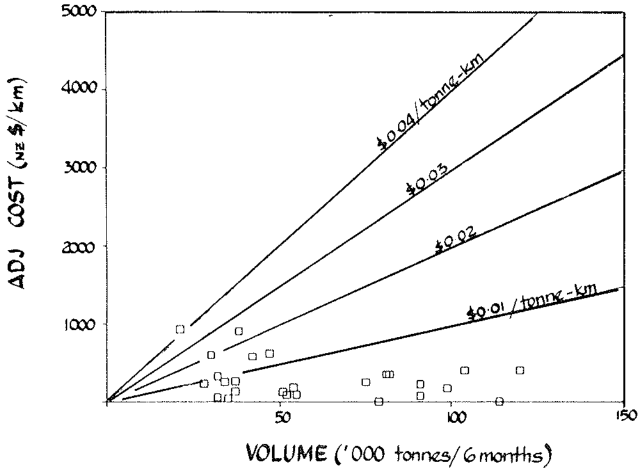

FIGURE 1 Relationship of cost and volume on Road 1 (sealed).

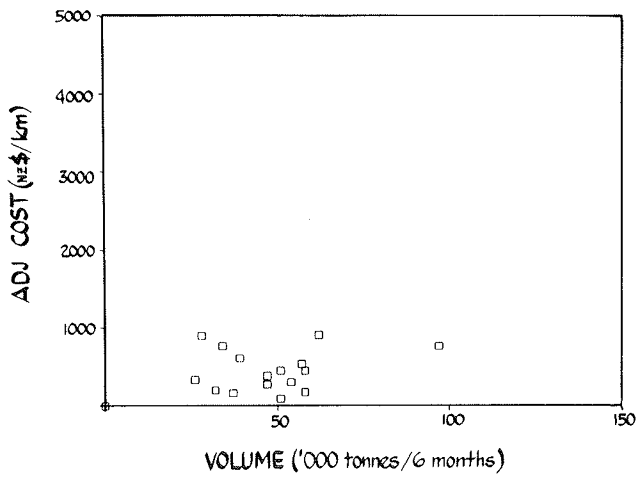

FIGURE 2 Relationship of cost and volume on Road 20 (sealed).

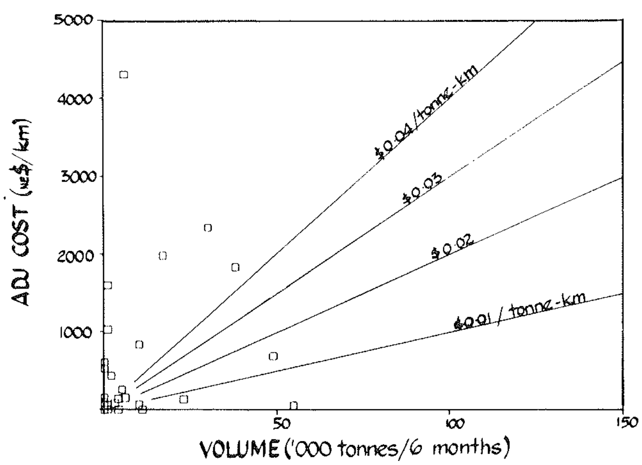

FIGURE 3 Relationship of cost and volume on Road 8 (unsealed).

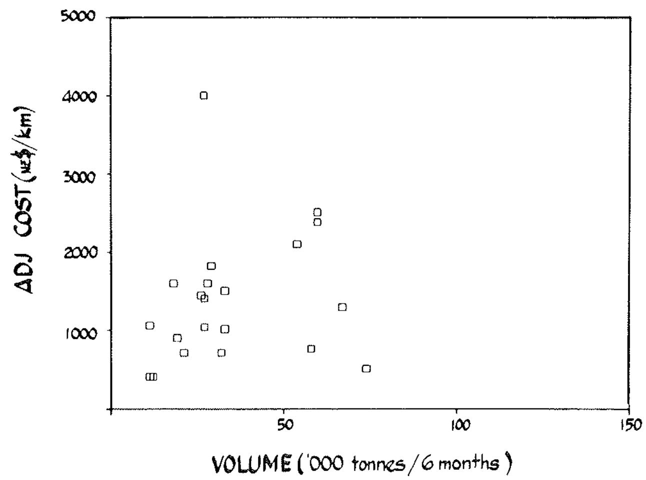

FIGURE 4 Relationship of cost and volume on Road 2 (unsealed).

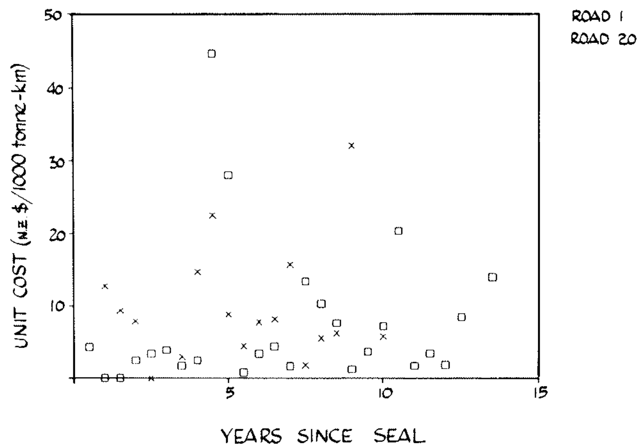

FIGURE 5 Relationship of unit cost and time on sealed roads.

Application to Other Areas

Maintenance costs will vary throughout New Zealand but the divergence between sealed and aggregate road costs will remain. Because the region studied has a reasonable supply of low-cost aggregate, it is probable that there will be a greater divergence in other areas.

When an attempt is made to apply these results to other areas, it is important to analyze the basic maintenance cost components, such as routine grading, routine gravel application, culvert and watertable maintenance, marker posts, and grass and vegetation control. These cost components should be analyzed on a rational basis as a starting point for the application of volume- and time-related variable costs.

When the results of this study are used to estimate road maintenance or to analyze and plan costs, an allowance should be made for both the initial sealing and periodic resealing and rehabilitation costs. These costs were not included in this study. An allowance can easily be made for both costs in a conventional, rational analysis or estimating procedure.

Comparison With Other Logging Road Maintenance Costs

Average road maintenance costs were obtained from two other forests for the sake of comparison. These are true road maintenance costs, gathered on an area basis, with 200,000 to 300,000 tonnes/annum/road. These costs vary between $NZ0.02 to $NZ0.035/tonne/km for annual maintenance and include both sealed and unsealed roads. The relationship of these costs to the analysis costs can be judged from the plotted lines of Figures 1 and 3.

Unfortunately, a wealth of information could have been available from these areas over a long period of time if costs had been kept on a road-by-road basis instead of an area basis.

ROAD MAINTENANCE RESEARCH

The NRB and LIRA are jointly undertaking funded, detailed trials into road maintenance for both sealed and unsealed roads.

Sealed Road

The sealed road trials are in two sections on a conventional, chip-sealed road. The first section is on a public road and has normal highway weight restrictions. The weight restriction for a single axle/twin tire is 8200 kg; that for a tandem axle/twin tire, at 1.2 m spacing, is 14 500 kg. The restriction on gross vehicle weight is 39 000 kg.

Monitoring of this section commenced in June 1986 and is expected to run for at least 5 years. The second section is an off-highway logging road. Although it currently only carries highway-weight road traffic, a strong possibility exists that this will be increased by a factor of 1.25 to 1.4 in the future.

The major work at this stage of the trials consists of the regular monitoring of roughness by NAASRA meter reading; Benkelman beam deflections in both summer and winter; pavement surface condition (visual rating and groundwater observations); maintenance effort and cost; climate records; and total heavy transport tonnage by way of weighbridge tallies for all heavy vehicles. A total of approximately 200,000 (net) tonnes is expected each year. Light vehicles can be estimated with sufficient accuracy from spot checks. Accurate records of construction methods are at hand.

Unsealed Road

This trial commenced in November 1986 and is expected to run for at least 2 years. The main work at this stage of the trial consists of the regular monitoring of roughness by NAASRA meter readings; Benkelman beam deflections in both summer and winter; pavement surface condition (visual rating); maintenance effort and cost; total heavy transport from weighbridge records; dust loss; aggregate loss; climate records; and total heavy vehicles by weighbridge tally. Loads heavier than those permitted on the highway may also be carried in the future.

Both of these trials provided an opportunity for the NRB, LIRA, and the local road controlling authority to work together on a project of mutual interest.

FURTHER RESEARCH

In 1984 and 1985, LIRA undertook an industrywide review of logging road standards, methods, and techniques. The review was discussed at a workshop of selected participants and the following research needs and directions were defined. Most of the research work is of a practical nature and is expected to produce immediate returns by the industry that oversees and directly funds the work.

Education and Training

LIRA's work in this field is directed toward encouraging people with civil engineering knowledge and training into the industry, and providing technical support to those who are already in the industry and have roading responsibilities. These responsibilities may often be shared with forestry, logging, and transport functions. There are signs at the university level that cooperation between the forestry and engineering schools will eventually strengthen engineering skills within the industry.

Effective Extension

The effective extension of information that is already available has been identified as a most necessary and productive endeavor. In early 1986, LIRA began distributing two, brief 4- to 6-page publications on a more or less monthly basis. *Road Notes* covers topics of interest to roading specialists in the forest industry (*4*). *Road Books* covers available books, pamphlets, and manufacturers' literature that may be of interest to roading specialists in the forest industry.

A booklet on compaction has also been distributed, and various computer programs have been made available to the industry. These programs cover machinery costing, estimating construction costs, network analysis, and the geometrics of vertical and horizontal curves. Although little of this material is original, its dissemination is believed to be most productive. Reaction from the 45 people on the mailing list has been enthusiastic. The 4- to 6-page format seems to be appropriate. Further work on the production of a forest roading terminology guide is proceeding well.

Compaction

Little compaction is performed on most forest roads, unless they are to be sealed. Adequate compaction, and appropriate gravel specifications that allow a clay binder fraction, would yield the following improvements: reduced maintenance grading requirements, reduced corrugation or washboarding, reduced potholing, and reduced infiltration of water (*5*). A clay-bound aggregate is often a cheaper roading material than an inappropriate, though high-quality, aggregate.

Compaction data from various published reports and trials can serve as a basis for practical requirements on forest roads (*6, 7*).

Soil Testing

Emphasis has been placed on simple tools to measure soil strength that preferably correlate with the traditional CBR tests. The Scala penetrometer and the Clegg Hammer have both been studied and used (*8, 9*). They can be used in most forest roads in which the full CBR testing procedure simply cannot be justified. The recent Technical Recommendation on soil testing that was published by the National Roads Board has been well-received (*10*).

Pavement Design

Emphasis has been placed on the extension of existing pavement design techniques and information to field engineers. Standard

New Zealand highway design techniques are not always the best economic choice for forest roads, but because little other information has been readily available, they have been used by default (*11*).

The various design methods and techniques of the U.S. Forest Service are being evaluated for use in the New Zealand forest industry together with other research that has appeared in various publications (*12-14*).

Alternate Log Transport Options

The potential use of six-wheel-drive trucks by civil engineers on steep roads is being investigated. The initial research is limited to a quick economic study, in which manufacturers' data and known cost and topographic information are being used. The results are promising enough to suggest that six-wheel-drive trucks and roads constructed with steep grades and minimum width may become part of New Zealand's logging transport scene. The limitations of double-handling at the transfer yard, and reservations about the size of the cartage must be overcome. The break-even point for this option to be economically feasible varies with the terrain and the transport distance. Each use of the technique also needs to be individually evaluated. This is not difficult with the ready availability of spreadsheet modeling and network analysis techniques on personal computers. A major limitation may be the difficulty of transporting conventional truck- or track-mounted yarders to the necessary landing sites on such roads (*15*). Another advantage of the type of road used in this alternate transport system is that trees can be planted in the road again until the next harvesting operation.

Transportation

In addition to the work outlined in the roading field, LIRA is performing work in the trucking field. Particular areas of interest are vehicle simulation by personal computer; vehicle gradeability in traction-limiting conditions; truck and road interaction, and the effects of road geometrics on vehicle performance; vehicle dimensions, legal limits (both weight and size), load securing, and cab guards; and extension of safe, sound techniques to the industry.

Seminars

The major forum for the presentation of project work and studies is at LIRA's annual seminar. The topic of the June 1987 seminar was "Logging Roads and Trucks."

The budget for roading and transportation is 25 percent of LIRA's total budget of NZ$900,000. The effective use of this budget is of paramount importance. Although fundamental research is useful and stimulating, the main emphasis must be to ensure that the industry gets value for its money. Industry contributes 50 percent of the total budget directly and this proportion is expected to increase over the next 3 years to possibly reach 70 percent of the budget.

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Information Deficiencies on Low-Volume Rural Roads

ROBERT S. HOSTETTER AND KENNETH W. CROWLEY

A description is provided of portions of a Federal Highway Administration study. The objectives of the study were to identify driver information needs on two-lane rural highways; identify potential driver problems that can be alleviated by way of low-cost information treatments; and develop a simple procedure that can be used by state and local personnel to identify information deficiencies on low-volume roadways for which adequate accident data are not available. As part of this study a 5,000-mi, 15-state sample of roadway and informational characteristics was obtained by use of a microprocessor-based, instrumented vehicle. The data base provided insights into the nature of existing informational problems and, by way of extrapolation, an estimate of the magnitude of informational deficiencies. Estimates are presented of the nationwide scope of informational problems at horizontal curves, narrow bridges, and intersections, which are the three primary features of the two-lane rural roadway system that are associated with higher accident rates. The field data collection and problem identification activities provided the basis for the development of a simple, in-vehicle procedure that can be used by state and county engineers, road supervisors, and others to identify problem sites and determine the appropriate remedial informational treatments. The procedure involves the consideration of driver expectancies; a form of commentary driving to identify problem sites; and the application of situation-specific information deficiency checklists to determine the appropriate remedial treatment.

Two-lane rural highways comprise the bulk of the nation's total roadway system. They represent about 80 percent of all roadways, both paved and unpaved, and nearly all (97 percent) of the total rural highway mileage. About 80 percent of these roads have low traffic volumes, with an average daily traffic (ADT) of less than 400 vehicles per day (vpd).

A recently completed FHWA study by Smith et al. indicated that approximately 34 million motor vehicle accidents can be expected to occur over the next 20 years on two-lane rural highways unless positive action is taken to correct the problems responsible for many of these accidents (*1*). The study also indicated that the likelihood of an accident on two-lane rural highways is greatest at horizontal curves, bridges, and intersections. Low-cost safety improvements such as signing and delineation were found to offer the greatest potential for cost-effective countermeasures on the two-lane rural road system.

The motor vehicle accident estimate of 34 million must be of concern to all those responsible for operating and maintaining this system because each accident represents a potential tort action. As was stated in a study by Oliver:

> ...the state is not an insurer of the highway, but it must provide a reasonably safe driving environment for a reasonably prudent driver. Inspection, advance notice, and correction of discovered defects all are within the zone of reasonable program activities. Ignorance of a defect or a dangerous situation—or knowledge of such a situation without warning the motorist to its presence—will expose the state to a liability judgement. A dangerous situation that remains extant for a prolonged period of time will leave the state in an indefensible situation (*2*).

A recent study of tort liability in the Pennsylvania Department of Transportation by Gittings indicated that signing deficiencies, such as missing, obscured, or inadequate signs, were the contributing factors that most frequently occurred in tort claims (*3*). These were the very types of problems that were found by the FHWA study to offer the greatest potential for cost-effective solutions.

This study is based on the results of a study by Hostetter et al. (*4*). It was designed to complement the efforts of the previously cited work by Smith et al. The three specific objectives of the study were to identify driver information needs on two-lane rural highways; identify potential driver problems that can be alleviated by way of low-cost information treatments; and develop an inexpensive procedure that can be used by state and county engineers, road supervisors, and others to identify information deficiencies.

The field survey portion of the study was intended to provide a basis for estimating the existing types and magnitude of information deficiencies. It was also intended to provide a detailed sample of the physical and informational characteristics of the two-lane rural roadway system. The latter objective was considered to be of extreme importance because relatively little was known about the existing roadway characteristics of the two-lane rural road system, particularly that part of the system that is under local control.

ROADWAY AND INFORMATIONAL CHARACTERISTICS

The field data collection effort employed the use of a vehicle that was equipped with sensors that provided information on distance, steering wheel position, accelerator position, and brake pressure. Sensor output was automatically recorded each second by a microprocessor and stored on disk. The system also enabled the manual input of codes that detailed specific roadway and informational characteristics.

Roadway characteristics and driver information systems were sampled on nearly 5,000 mi of open road sections of the two-lane rural road system in 15 states throughout the country. These data served as the basis for estimating information deficiencies on the nation's two-lane rural road system. They also provided descriptive data regarding roadway and informational characteristics.

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The sampling plan was developed to ensure that a broad range of geographic and terrain effects was represented. States were sampled in seven of the nine FHWA field regions in the continental United States. Approximately 58 and 42 percent of the sample mileage came from east and west of the Mississippi, respectively. Paved roadway in rolling and mountainous terrain was deliberately emphasized in the sample roadway mileage breakdown shown in Table 1. Unpaved roadways were not sampled extensively because they account for a very low percentage of total vehicle miles traveled. Furthermore, a large percentage of that travel is by familiar drivers; consideration of information deficiencies was therefore of a lower priority. Rolling and mountainous terrain were emphasized because of the relative difficulty of driving in these regions and the increased probability of encountering a variety of geometric situations that are likely to create driving problems.

The total paved roadway sample was classified in a manner consistent with that of the Smith et al. study, as shown in Table 2 (*1*). As expected, a general relationship exists between pavement width and curves/mi; wider pavements are more likely to be associated with fewer curves/mi, which is an indication of an overall higher design, and vice versa.

Roadway Characteristics and Features

Surface and Shoulder Characteristics

The sample was almost evenly split between roadways of 20 ft or less (46 percent) and those greater than 20 feet (54 percent). Ninety-five percent of the roadways were asphalt. This generally reflects published statistics.

The surface of nearly 85 percent of the road mileage sampled was in good condition as judged by the ability to comfortably maintain a speed near the speed limit on tangent sections or curves where horizontal alignment did not control the speed. Less than 1 percent of the road surfaces were judged to be in poor condition.

Roads with no shoulder of any kind accounted for about 36 percent of the paved roadway mileage sampled. Of the remaining 3,100 mi of paved roadway in the sample, 64 percent had unpaved shoulders and 36 percent had paved shoulders. Shoulders were coded as either two-wheel or four-wheel. A four-wheel shoulder was one that was wide enough to get a car completely out of the travelway. Nearly 66 percent of the roadways with shoulders had shoulders of a two-wheel width.

TABLE 1 SURFACE TYPE BY TERRAIN TYPE FOR TOTAL SAMPLE MILEAGE

	Surface Type		Total	
Terrain	Unpaved (mi)	Paved (mi)	Miles	Percent
Flat	28	468	496	10
Rolling	104	3,161	3,265	66
Mountainous	57	1,128	1,185	24
Total	189	4,757	4,946	100

TABLE 2 PAVED ROADWAY MILEAGE IN PAVEMENT WIDTH, INTERSECTIONS/MILE, AND CURVES/MILE CATEGORIES

PAVEMENT WIDTH	INTERSECTIONS/MILE							
	0-2.5				2.5-5.0			
	CURVES/MILE				CURVES/MILE			
	0-1.0	1.0-2.5	2.5-4.5	>4.5	0-1.0	1.0-2.5	2.5-4.5	>4.5
< 20	119	351	547	344	0	3	15	4
20	106	353	178	126	0	9	10	0
> 20	890	905	427	244	31	33	52	6
Subtotals	1115	1609	1152	714	31	45	77	10
TOTALS	4590				163			

Roadway Features

The roadway features of primary interest on the basis of highest accident probabilities are horizontal curves, narrow bridges, and intersections. The frequency of occurrence of these features is shown in Table 3 for three terrain types. The primary difference between the terrain types is the frequency at which curves occur.

Informational Characteristics

Pavement Striping Characteristics

Slightly more than half of the paved mileage had both centerlines and edgelines. Another 31 percent had only centerlines. Seventeen percent of the paved mileage sampled had neither edgelines nor centerlines.

As pavement width increased, the percentage of roadways with either centerlines or centerlines and edgelines also increased. This is reasonable because wider pavements generally imply a higher type of geometric design to serve greater traffic volumes, which in turn suggests the likelihood of a higher standard of marking.

General Signing Characteristics

A variety of specific regulatory and warning signs was coded in the data sample. The regulatory signs were SPEED LIMIT, REDUCED SPEED AHEAD, STOP, and YIELD. All major curve, intersection, and narrow bridge warning signs were identified, such as REVERSE CURVE and STOP AHEAD. A variety of warning signs that were considered to be less important to the requirements of the study, such as "Divided Highway," were simply coded "other." The SLIPPERY WHEN WET and BRIDGE FREEZES BEFORE ROADWAY signs were not coded at all because their presence or absence is a matter of agency policy. If an advance warning sign included an advisory speed plate, the speed advisory value was also coded in the data base.

The rate at which signs were encountered on various types of terrain is shown in Table 4. It was estimated that three regulatory or warning signs of some type could be found on each mile of the open road sections of the paved two-lane rural system, not including the two signs that were not coded.

INFORMATION DEFICIENCIES

Individuals need information to be able to drive safely, conveniently, efficiently, and comfortably. Most of the needed information is received visually from the driver's perception of the highway environment. The environment includes such formal information as signs and markings provided by a government jurisdiction, and natural information. Natural information refers to all other elements of the highway environment that assist the driver in making correct alignment and speed control decisions, such as tree or brush lines. Natural

TABLE 3 PAVED ROADWAY FEATURES BY TERRAIN TYPE

Terrain	Miles	Curves/ Mile	Narrow Bridges/ Mile	Intersections/ Mile
Flat	468	1.37	0.15	1.10
Rolling	3,161	2.10	0.16	1.24
Mountainous	1,128	4.30	0.11	0.99
All Terrain	4,757	2.55	0.15	1.17

TABLE 4 SIGN SUMMARY BY TERRAIN TYPE ON SAMPLE OF PAVED ROADS ONLY

TERRAIN	MILES	All Warning Signs/ Mile	Speed Signs/ Mile	Speed Reduction Signs/ Mile	STOP/ YIELD Signs/ Mile	Total Signs/ Mile
Flat	468	.72	.25	.04	.10	1.12
Rolling	3161	1.05	.32	.03	.07	1.49
Mountainous	1128	1.22	.34	.04	.02	1.63
All Terrain	4757	1.06	.32	.03	.07	1.49

information can be treated to enhance the visual cues in given situations. For example, roadside vegetation can be tapered to improve the way in which the natural information indicates the narrowness of a bridge.

Problem Overview

An information deficiency is considered to exist whenever specific information necessary to drivers about the road ahead is not provided. The need for any form of specific information usually results from a change in alignment or geometry that is not visible in the time needed for the driver to take appropriate action safely, such as a change in speed, an alteration in lateral placement, or a stop.

A number of distinct types of information deficiencies were observed in the field. They range from situations in which necessary information was not available to the driver because it was obscured, missing, or incomplete, to situations in which the information presented was misleading, confusing, or inconsistent.

Of all the types of information deficiencies that were identified, misleading information may be the most difficult to immediately identify because it can occur in a variety of ways. Misleading information occurs not only when the formal information available is incorrect or incomplete, but also when available natural information misleads the driver. An example of misleading information transmitted by signs is a location in which advance work zone warning signs are placed when construction has already been completed. An example of misleading natural information is a sight-restricting crest-vertical-curve in which trees or telephone poles beyond the crest appear to indicate that the road is straight when, in fact, a horizontal curve exists immediately after the crest.

An excellent example of a situation with misleading information that was observed in the field was a narrow bridge that had a centerline painted through it. The centerline provided an associative cue from past experience that the bridge was sufficiently wide for two vehicles to safely pass each other. However, the width from bridge rail to bridge rail was only 15 ft. This situation was further compounded because the reduced width could not be easily perceived by the driver because the bridge was situated on a slight reverse curve. The distance between Manual of Uniform Traffic Control Devices Type 3 object markers could lead an approaching driver to believe that the bridge is much wider than it actually is. If a driver does not perceive the reduced width and an oncoming vehicle attempts to cross the bridge, the driver might have to brake sharply before the bridge or, even worse, crash into the bridge abutment or the oncoming vehicle.

Horizontal curves were the most prevalent physical feature (over 12,000) in the nearly 5,000 mi of rural two-lane roads that were sampled. The most information is needed on horizontal curves, particularly isolated sharper curves. Narrow, width-restricted bridges, which represent only about 700 of all types in the data base, are a distant second. Stop- or signal-controlled intersections, which total slightly more than 300, are next in frequency of occurrence. Railroad crossings occurred so infrequently in the sample that they do not merit separate attention. Each of these three geometric features is discussed in terms of the magnitude of potential information deficiencies that was estimated from the sample.

Information Deficiencies Related to Curves

The accident data for curves indicate that both degree of curvature and frequency of curves influence accident frequency and severity. This provided the focus of the analysis of curve-related information deficiencies.

The total sample of curves recorded in the data base was distributed by curve category and advance tangent length. Three curve categories were defined: gentle (≤6°); moderate (6° to 10°); and sharp (>10°). A 6° curve roughly corresponds with a maximum design speed of about 55 mph, whereas a 10° curve corresponds with a design speed of about 45 mph. Two different advance tangent categories also were defined: nonisolated (<1,000 ft) and isolated (≥1,000 ft). The isolated category indicates that drivers approaching a curve are essentially free of any speed-reducing influences of immediately preceding curves.

Approximately 26 percent of all the curves in the sample were characterized as isolated. An isolated curve is of the greatest concern from the standpoint of curve-related information deficiencies. The long advance tangent of an isolated curve encourages travel at or above the roadway speed limit. The sharper the curve, the more a speed differential between the tangent and the curve is likely to occur and the more necessary it becomes to provide some form of advance warning information.

Information deficiencies are only presented for isolated curves with moderate and sharp horizontal alignments. Non-isolated curves, such as those with advance tangents of less than 1,000 ft, are considered less critical in terms of information deficiency problems. This is because a driver is more likely to be cautious or drive more slowly when entering a curve if it has been closely preceded by one or more curves instead of a long tangent section.

A total of 1,386 curves in the data base met the joint criteria of being isolated and sharp; in other words, they had a degree of curvature of at least 6°. This represents slightly more than 11 percent of the total curve data base. Information on the number of isolated sharp curves in the data base and whether or not they had advance curve warning signs can be found in Table 5. Approximately 43 percent of all such curves have an advance warning sign of some type. As might be expected, the percentage of isolated curves with advance warning signs increased with an increase in the degree of curvature, which demonstrated that the need to warn drivers under these conditions was clearly recognized.

Recognition of the potential difficulty of driving on horizontal

TABLE 5 DISTRIBUTION OF MODERATE TO SHARP ISOLATED CURVES BY PRESENCE OR ABSENCE OF ADVANCE CURVE WARNING SIGN

	Degree of Curve		
	6°-10°	>10°	Total
Advance Warning Provided			
Number	302	298	600
Percent	(40.5)	(46.6)	(43.3)
No Advance Warning Provided			
Number	444	342	786
Percent	(59.5)	(53.4)	(56.7)
Total	746	640	1,386

curves on two-lane rural highways was evidenced by the extent of information that was provided. Two-thirds of the sharp curves that had advance curve warning information also had advisory speed information.

All of the isolated moderate or sharp curves that had no form of advance warning were considered to be locations with potential information deficiencies. According to that criterion, 786 curves in the data base could be considered deficient. Although this number represents only about 6 percent of all curves identified in the data base, it represents more than half of all isolated moderate and sharp curves.

One cannot conclude from this that more than half of all moderate and sharp isolated curves are information-deficient. Whether or not a curve with no advance warning can be considered potentially information-deficient depends on the approach speed on the tangent and the speed that can be safely maintained around the curve. Tangent approach speeds available in the data base were examined for each of the 786 unsigned, isolated, moderate to sharp curves.

There are 193 curves in the data base for which the relationship between approach speed and degree of curvature indicates a potential information deficiency. Less readily definable are another 169 moderate (6 to 10°) curves with approach speeds of 45 to 54 mph. Some of these curves may also have approach speed and degree of curvature relationships that could create a potential information deficiency. Because of the manner in which speeds and curve degree were distributed in the data base, it was assumed that 90 percent of these 169 curves did not require specific curve-related information. Therefore, 210 of the 786 isolated unsigned curves in the data base were assumed to be information-deficient. This represents slightly more than one out of every four of the unsigned, isolated, moderate to sharp curves in the total field data sample. This translates to about one potentially information-deficient curve for every 23 mi of paved roadway in the data sample.

Information Deficiencies Related to Narrow Bridges

For the purposes of this study, any constriction that included a vertical structure was considered to be a narrow bridge. Although most of the structures so identified were at least 20 ft in length (the standard definition of a bridge), some could be more accurately identified as culverts. In either case, however, a narrowing of cross-section and a vertical structure with which the driver could collide were present. Regardless of the terminology used, similar safety problems are likely to benefit from similar informational treatments.

According to this definition of a narrow bridge, the number of such constrictions was much greater than the 60,000 reported by Ivey et al. (*5*). Of the 701 narrow bridges and constrictions found in the 5,000-mi sample of roadways, it was estimated that one such site existed every 7 mi. When only the estimated 1.6 million mi of paved rural two-lane roadway are considered, the total number of narrow bridges and constrictions is estimated to be greater than 200,000, which is over three times that reported by Ivey et al.

Less accident information exists for bridge and culvert sites than for curves, intersections, and other road situations. However, based on a review, Smith et al. estimated that approximately 20,000 reported collisions occur annually at such constrictions (*1*). Study results from Kihlberg and Tharp suggest that the presence of a narrow bridge may increase the probability of an accident by up to 2.5 to 3 times the base rate on a vehicle exposure basis (*6*).

Although limited data exist regarding the severity of accidents at narrow bridges, Perchonok indicated that more than 50 percent of the accidents at constrictions involved fatalities (*7*). This high percentage of fatalities merits, at the very least, adequate informational treatments at narrow bridges regardless of traffic volume. The use of hazard panels, advance warning (when needed as a result of a sight distance restriction), and advisory speed signs formed the focus of the deficiency analysis.

Bridges were defined as being narrow if a shoulder width decrease, shoulder loss, or pavement width decrease was observed. In terms of driver safety, locations that were characterized by a pavement decrease were considered to be the most serious. Total shoulder loss, although generally less hazardous than an actual pavement decrease, can indicate potential problems, particularly when the pavement width is minimal, such as 20 ft or less. The shoulder width decrease category represents the least potentially serious of the three narrow bridge groups. It is included, however, because any reduction in the total width of pavement and shoulders could present a problem, particularly when sight distance is restricted and the necessary advance or at-bridge warning information is not available.

Nearly three-quarters of the 701 narrow bridges in the data sample were identified as being sight-restricted; for example, a horizontal or vertical curve existed within a specified (speed-dependent) distance in advance of the narrow bridge. Approximately 85 percent of the sight-restricted bridges had no advance warning. Only about one in four locations in which an advance warning existed had an associated speed advisory. This represents less than 4 percent of the total sample of sight-restricted narrow bridges, which indicates that this form of supplemental information was perceived as unimportant, at least in terms of a width-constrained situation.

The fact that over 85 percent of the sight-restricted narrow bridges had no advance warning signs is not meant to suggest that the overwhelming majority of narrow bridges on the two-lane rural road system have information deficiencies. It would be equally erroneous, however, to assume that all of these locations are free of any information deficiencies. How then can one reasonably estimate the magnitude of the potential information deficiency problem? It is important to recognize that ADT was low on a large proportion of the rural roads surveyed. This significantly reduces the probability that opposing vehicles will come into conflict at the narrow bridge, and reduces the need for advance warning signing.

It is necessary, however, to identify to the oncoming driver the existence and location of any lateral obstruction created by the narrow bridge, such as raised curbs or a bridge rail. This can be accomplished through the use of object markers. Nearly 80 percent of the sight-restricted bridges had some form of object marker.

The sight-restricted narrow bridge sample is broken down in Table 6 by type of width restriction, presence or absence of some form of object marker, and presence or absence of some form of advance warning signing.

Because the two-lane rural road system tends to be composed of roads with comparatively narrow travelways, a total loss of shoulder and certainly an actual travelway decrease pose potential problems for drivers. It is important to note that although 49 percent of the shoulder-loss type of narrow bridges were located on roads with travelways of 20 ft or less (near the

TABLE 6 DISTRIBUTION OF SIGHT-RESTRICTED NARROW BRIDGES BY TYPE OF WIDTH RESTRICTION, PRESENCE/ABSENCE OF OBJECT MARKER, AND PRESENCE/ABSENCE OF ADVANCE WARNING SIGN

	SHOULDER DECREASE		SHOULDER LOSS		PAVEMENT DECREASE	
	OBJECT MARKER		OBJECT MARKER		OBJECT MARKER	
ADVANCE SIGNING	YES	NO	YES	NO	YES	NO
YES	7	1	20	1	29	16
NO	132	45	192	37	25	12
TOTALS BY COLUMN	139	46	212	38	54	28
TOTALS BY RESTRICTION N (%)	185 (35.7)		250 (48.4)		82 (15.9)	

sample average), 75 percent of all the pavement-decrease type of narrow bridges were located on roads in which the pavement widths were 20 ft or less. Therefore, an already narrow roadway condition is exacerbated at the narrow bridge.

It is indicated in Table 6 that almost 16 percent of the sight-restricted narrow bridges are of the pavement-decrease type. Nearly 15 percent of these were observed to have neither advanced warning nor any form of at-site object marker. The total absence of information on a narrow roadway with a further travelway decrease is a reason for concern regardless of whether or not a sight restriction occurs. When a sight restriction does exist, the potential for serious safety problems also exists.

Although the absolute number of such worst-case problem locations is small (12 in 4,757 mi of paved roads), an extrapolation to the entire two-lane rural road system suggests the possible existence of over 4,000 such locations. Although the total absence of warning information is potentially the most serious problem, it is not the only one associated with narrow bridges. Besides the 12 pavement-decrease type of narrow bridges that have neither advance warnings nor object markers, another 16 pavement-decrease sites exist with advance warning signs that have no at-bridge object markers. Although the lack of object markers may not be a significant problem in daylight hours, it is a matter of great potential concern at night, when drivers get little advance visual information about lateral clearance at the narrow bridge. The 37 instances of sight-restricted bridges with complete shoulder loss and with no advance warning or object marking can similarly pose additional safety hazards to drivers.

In all, 18 percent of the sight-restricted narrow bridges in the data sample, regardless of degree of constriction, were information-deficient in that both signs and object markers were missing. Potential safety problems related to the absence of object marking on narrow bridges are not limited to sight-restricted locations. The absence of object marking on narrow bridges in which a pavement decrease or total shoulder loss exists could pose safety problems, particularly at night.

State and local agencies may be leaving themselves open for tort claims in the event of accidents, because the courts may well hold that a reasonably safe driving environment was not provided. It appears that some type of object markers should be installed on all narrow bridges to help oncoming drivers position themselves in the lane to avoid bridge-related obstructions. Use of an advance warning sign appears to be more directly associated with higher ADTs, in cases in which the probability of meeting an oncoming driver on or adjacent to the narrow bridge increases significantly, or in cases in which a sight restriction is close enough to the bridge that use of object markers alone is insufficient.

Information Deficiencies Related to Stop - or Signal-Controlled Intersections

The multiple-vehicle intersection accident in which a driver fails to stop is of primary concern when one considers information deficiencies. According to Smith et al., angle accidents are the predominant multiple-vehicle type; they constitute 60 percent of all fatal, two-lane rural intersection accidents and 80 percent of all such multiple-vehicle accidents (*1*). These accidents are essentially caused by a driver on the controlled approach proceeding into the crossing roadway without appropriate clearance. When the failure to stop primarily results from inadequate warning of a sight-restricted approach, as opposed to a willful act, the intersection is amenable to information-related correction. The focus of this intersection-related information deficiency analysis was on sight-restricted approaches to intersections that are controlled by stop signs, where 50 percent of the rural intersection accidents are reported to occur (*1*). The analysis also considered whether or not advance warning was provided in these situations.

The total data base contained approximately 5,600 intersections all of which were on roadway sections outside any type of urban area, such as a village, town, or city. Only 321 stop-controlled intersections and 64 signal-controlled intersections were identified. This represents about 7 percent of the total number of intersections traversed. Slightly more than 25 percent of these 385 intersections were sight-restricted; in other words, they did not have an adequate stopping sight distance. Although these sight-restricted intersections represent less than 2 percent of all intersections observed, they merit attention because the potential for accidents is great in these situations. If a sight restriction causes a driver to enter an intersection improperly, any right-angle accident that results could be extremely serious.

Fifty-seven percent of the sight-restricted, stop- and signal-controlled intersections had no advance warning signs. The sight restrictions were about evenly split between crest vertical curves and sharp (>10°) horizontal curves.

In nearly 60 percent of the cases in which stop or signal control was deemed necessary and a sight restriction occurred, no advance warning sign was provided. The potential severity of accidents at such problem locations indicates that jurisdictions need to carefully examine their rural stop- and signal-controlled intersections, however few in number they may be, to ensure that adequate advance warning is being provided and adequately maintained. Omission of this inexpensive but necessary corrective action could result in serious accidents, and tort liability action could be initiated against the responsible governmental agency.

NATIONAL IMPLICATIONS OF THE FINDINGS

A variety of potential information deficiencies was identified in the nearly 5,000 mi of two-lane rural roadway that were sampled. Extrapolation of the sample results to the 1.6-million-mile paved portion of the two-lane rural roads in the United States yields an order of magnitude estimate of the total potential information deficiency problems. The estimated average frequency of each of the deficiencies is shown as follows.

Feature or Deficiency	*Estimated Average Frequency*
Isolated, horizontal, moderate to sharp curves with no advance warning	One every 30 mi
Sight-restricted narrow bridges with no object markers or advance warning	One every 51 mi
Sight-restricted, stop- and signal-controlled intersections with no advance warning sign	One every 79 mi

Because the 5,000-mi data sample emphasized rolling and mountainous terrain, it almost certainly contains more curves than would be the case if a more representative sample had been obtained. A comparison of the total rural mileage by terrain (31.5 percent flat, 58.9 percent rolling, and 9.6 percent mountainous) with the data sample collected as part of this study (9.8 percent flat, 66.5 percent rolling, and 23.7 percent mountainous), and the average curves/mi by terrain type obtained, suggests that the estimated overall frequency of potential horizontal curve problems might be more on the order of 1 every 30 mi, as was shown earlier, as opposed to the rate of 1 every 23 mi that was reported in the field data.

If the three estimates are grouped, one might expect about 66 potential information deficiencies, irrespective of type, every 1,000 mi of paved roadway. Use of this estimate and an assumed 1.6 million miles of surfaced, two-lane rural roads indicates that over 100,000 potential information deficiencies may exist. Horizontal curves account for approximately 53,000 of the locations; sight-restricted narrow bridges account for another 31,000; and sight-restricted, stop- and signal-controlled intersections account for the remaining 20,000 locations.

Although the potential problem nationwide is great, information deficiencies exist mostly on systems that are under the jurisdiction of a myriad of local agencies, each of which may have comparatively few specific problems. This situation is not amenable to direct federal action. It is a problem that must be addressed indirectly through technology transfer to sensitize responsible local agencies and governments to the existence of the problem, provide simple, effective techniques to identify information-deficient locations, and suggest simple, inexpensive solutions. An example of such a procedure is briefly described in the following paragraphs.

A simple procedure that can be used by state and local agencies to identify information deficiencies was developed as part of this study (*8*). It essentially consists of a windshield survey that requires no equipment other than a tape recorder. The procedure is largely based on the concept of identifying driver expectancy violations and uses a commentary driving process to identify potential problem sites. It also includes situational problem checklists that can be used in a more detailed analysis of the identified potential problem sites. The checklists aid in the development and specification of the most appropriate countermeasures. An example of one of the checklists is shown in Figure 1. The procedure is described in a study by Hostetter et al. (*8*).

Although a relatively simple survey technique is employed, its use in a programmatic manner can materially simplify problem identification. As such, it should enable local jurisdictions to identify and correct high-risk locations and thus reduce the likelihood of being sued. Such a program may also be of value in cases in which tort claims are instituted. In a study that was cited earlier, Oliver provided an example of a case in which the programmatic use of a similar type of windshield survey resulted in the reversal of a decision in a case that a state agency had initially lost in a lower court (*2*).

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INFORMATION DEFICIENCY EVALUATION
NARROW/ONE-LANE BRIDGES

PART I

ROUTE ID ________________ LOCATION: ________________ MILES FROM REFERENCE POINT ________________

APPROACH DIRECTION N S E W (circle)

DATE ________ TIME ________ AM PM INSPECTORS ________________

APPROACH SPEED DURING SURVEY ________ MPH

SPEED LIMIT: a) Posted ________ MPH or b) Estimated ________ MPH (one entry)

DECISION SIGHT DISTANCE (circle one set)

SPEED (max of above)	30	35	40	45	50	55	60
DSD (feet)	230	290	355	430	510	590	680

PART II

(1) Is the bridge clearly visible from decision sight distance?
____ Yes ____ No

If no, go to (3)

(2) From decision sight distance, can you perceive the reduced roadway width at the bridge?
____ Yes ____ No

If yes, go to (5)

(3) Is there a NARROW BRIDGE or ONE-LANE BRIDGE warning sign present?
____ Yes ____ No

If no, go to (5)

(4a) Is the warning sign accurate? (i.e., the ONE-LANE BRIDGE is applicable to bridges with usable roadway widths less than 16 ft or 18 ft if a significant number of wide vehicles cross the bridge or if the approach alignment is winding)
____ Yes ____ No

(4b) Is the warning sign clearly visible on the approach?
____ Yes ____ No

(4c) Is the warning sign properly designed according to the specifications in the MUTCD?
____ Yes ____ No

(4d) Is the warning sign properly located? (i.e., neither too far upstream such that you would "forget" it or too close to the bridge such that you still would not have sufficient time to select a safe speed and decelerate to it) (Check Table of Placement Distance for Advance Warning Signs in MUTCD)
____ Yes ____ No

(4e) Is there a supplemental speed advisory plate attached to the warning sign?
____ Yes ____ No

(5) Do other informational sources (i.e., hazard panels, guardrails, edgelines, roadway edges, bridge abutments, etc.) provide information suggesting 1) that the situation ahead is **not** a narrow/one-lane bridge, 2) that usable roadway width across the bridge is wider than it actually is, or 3) that a narrow/one-lane bridge is located further downstream?
____ Yes ____ No

If yes, then identify those sources and describe how they provide confusing, conflicting or misleading information:

____
____
____
____

(6) Is the sight distance to opposing vehicles sufficient for you to make a safe decision on whether you can safely cross the bridge and to safely execute the selected maneuver?
____ Yes ____ No

(7) Is the presently available information sufficient for you to recognize the narrow/one-lane bridge at a distance such that you can decelerate safely to a safe and comfortable crossing speed?
____ Yes ____ No

(8) Would the presently available information be sufficient for you to recognize that a narrow/one-lane bridge is downstream:
- during nighttime conditions? ____ Yes ____ No
- when the roadside vegetation is at its densest growth? ____ Yes ____ No

FIGURE 1 Sample problem location checklist.

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Development of Geometric Design Standards for Low-Volume Roads in Canada

D. BEWS, G. SMITH, AND G. TENCHA

Approximately 76 percent of the road system in Canada has been classified as rural local roads that carry low traffic volumes. In the past, a uniform set of geometric design standards for roads was not available in Canada. The lack of national standards for low-volume roads resulted in agencies developing their own. These standards may not have been compatible with the required function of the road and also had the effect of nonuniform treatment of roads between road jurisdictions. Transportation planners and designers were faced with the problem of reducing national standards, which were originally developed for a higher classification of roads, to meet economic constraints. It became evident that there was a need to find ways to construct these roads more economically and to maintain their safety and effectiveness. As a result, the Roads and Transportation Association of Canada initiated a project to develop a national set of geometric design standards for low-volume roads. A separate chapter for low-volume roads is now included in the *Manual of Geometric Design Standards for Canadian Roads*. A discussion is presented of the approach used to develop the geometric design standards for low-volume roads, the results and findings, and future research that should be performed to further refine the standards.

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The Canadian Road Network consists of over 800 000 km of roads that serve a population of approximately 25.6 million. Approximately 610 000 km, or 76 percent, of these roads can be classified as rural local roads that carry low traffic volumes. In addition, 490 000 km of these rural local roads have either earth or gravel surfaces.

In the past, geometric design standards for these types of roads were not specifically addressed in Canada. Both road planners and designers were faced with either using national standards that were developed for a higher classification of roads, which resulted in roads being built at a great cost that was unrelated to their function, or reducing these higher classification standards to meet economic constraints, usually without a logical basis for doing so.

In many instances, the lack of national design standards for these roads and the pressure to reduce costs resulted in agencies developing their own design standards or, in certain instances, in constructing roads without regard for any design standards. This has resulted in the creation of standards that are not compatible with the road function, nonuniformity of standards between jurisdictions, arbitrary selection of standards, and in many cases an unsafe road.

It was therefore evident that there was a need within the Canadian road system for a set of national geometric design standards that recognized the unique qualities of these rural roads with low traffic volume.

The Roads and Transportation Association of Canada (RTAC) is a nonprofit, nonpartisan association of 600 corporate members including federal, provincial, territorial and municipal governments; a wide range of carriers and suppliers of transportation goods and services; and the academic community. In 1983, RTAC approved the establishment of a project steering committee to research and develop a set of geometric design standards for low-volume, rural roads that would be the product of a consensus of the majority of users in Canada.

These standards would be incorporated as a separate chapter in the RTAC *Manual of Geometric Design Standards for Canadian Roads*, which was currently in the process of being updated (*1*). The objectives of the project were defined as follows:

- To establish uniform national standards for the classification of low-volume roads to meet the special services requirements of road agencies across Canada,
- To provide standards compatible with the present economic requirements without jeopardizing the safety or effectiveness of the road, and
- To provide standards for road agencies that relate to the type of road function and that will ensure standardization.

The project steering committee, which consisted of representatives from federal, provincial, territorial, and county road authorities, and the private sector, established terms of reference and selected a consultant to perform the work. Funding for the project was provided by the Council on Highway and Transportation Research and Development (CHTRD) of the Roads and Transportation Association of Canada and the Federal Department of Indian Affairs and Northern Development.

METHODOLOGY

A literature review was undertaken to determine present practices in Canada, the United States, and selected foreign countries and to identify available, related research data.

Existing design standards used by the various Canadian federal, provincial, and municipal agencies, and agencies in other countries were reviewed and documented.

A questionnaire was developed and distributed to a representative sample of Canadian road agencies, and to private companies that were engaged in resource development. The questionnaire included a number of questions related to draft design standards to determine the reaction of potential users. It also included questions designed to obtain opinions, suggestions, experience, and comments related to low-volume roads to assist in establishing the design standards.

Design standards were developed for low-volume roads through a synthesis of existing standards that were in use by the various road agencies in Canada. Adjustments were made when appropriate, based on standards used by other agencies and available research.

Low-volume roads in Canada were defined by the Committee as roads with an average daily traffic (ADT) of 200 vehicles or less, the service functions of which were oriented to rural road systems, roads to or within isolated communities, recreational roads, and resource development roads.

DISCUSSION OF STANDARDS

Classification

A separate classification system was developed for low-volume roads that recognized their unique characteristics and function or use. The system enabled the designer to select a set of geometric design standards that were related to the use of the road and were economically compatible with the low volumes. The system was based on service function, traffic volume, and design speed.

Service Function

Because low-volume roads serve different functions, and in order to address their different design requirements, the roads were divided into the following three functional categories:

- Rural road systems and roads to or within isolated communities,
- Recreational roads, and
- Resource development roads.

These functional categories reflect the differences in traffic and land service that influence the selection of design standards, particularly roadway width. The three categories, which are similar to the categories used in the new design manual of the American Association of State Highway and Transportation Officials, were confirmed through the questionnaire as being appropriate for low-volume roads in Canada (*2*).

Rural roads and roads to or within isolated communities provide access to farms, residences, and businesses or other abutting properties. Traffic consists of light and medium vehicles with occasional heavy trucks.

Recreational roads provide access to and within all types of recreational areas. Traffic generally consists of cars, trailers, camper-truck units, and maintenance vehicles. Recreational roads are further subdivided into primary roads, perimeter roads, and internal roads, which essentially reflect differences in expected operating speeds. This category is similar to the classification system for recreational roads of both Parks Canada and AASHTO (*2, 3*).

Resource development roads include all resource-related roads such as forest roads, mining roads, and roads required for energy development. Traffic on these types of roads is predominantly large, heavily loaded trucks.

Traffic Volume

An ADT of 200 vpd was selected as the maximum volume for which the design standards are intended. This was based on the fact that the majority of low-volume roads in Canada have traffic volumes below this value, and was confirmed through responses to the questionnaire. The design standards satisfy safety requirements for an ADT of up to 200 vpd.

The ADT value is used for design instead of Average Annual Daily Traffic (AADT), which is normally used in the design of the higher classification roadways, to account for the variation in traffic volumes that can be expected due to seasonal use. The ADT is defined as the total volume of traffic during a given time period, in whole days, greater than 1 day and less than 1 year, divided by the number of days in that period. Other road agencies that use ADT for design traffic volume include AASHTO, the U.S. Department of Agriculture Forest Service, and the Zambia Roads Department in Africa (*2, 4, 5*).

The current ADT is established as the design ADT if low growth is expected. If higher growth is expected, the projected 10-yr ADT is used as the design ADT. If the design ADT is greater than 200 vpd, the designer must use the design standards for the higher classification road.

As is the case with roads of higher classification, traffic volumes do not directly influence design standards for sight distance, horizontal alignment, or vertical alignment. They do, however, influence road cross-section elements.

Road cross-section elements were developed for the following:

- Two-lane roads for ADTs less than 100 vpd, and for ADTs between 100 and 200 vpd;
- One-lane, one-way roads for ADTs up to 200 vpd;
- One-lane, two-way roads for ADTs up to 50 vpd; and
- One-lane, two-way resource development roads for ADTs up to 150 vpd.

The Average Daily Truck Traffic (ADTT) also influences roadway widths. When the ADTT is greater than 15 vpd, roadway widths are increased. This was based on the research presented in a study by the National Cooperative Highway Research Program (*6*).

Design Speed

The design speed concept is used to select design standards for low-volume roads. The design speed ranges for low-volume roads are shown in the following table.

Service Function	*Design Speed (km/h)*
Rural road systems and roads to or within isolated communities	30-100
Recreational roads	
Primary	30-100
Perimeter	30-50
Internal	30-50
Resource development roads	30-100

Design speeds higher than 100 km/h for low-volume roads were not considered justifiable in terms of the cost of meeting the higher design standards. Design speeds of 50 km/h or less are recommended for perimeter and internal recreational roads to satisfy environmental constraints and aesthetic considerations. Design speeds of 50 km/h or less are recommended for one-lane, two-way roads in the interest of safety.

The most important factors considered in selecting design speed include terrain type, trip length, and service function. Lower design speeds are considered appropriate in rolling or mountainous terrain because of horizontal and vertical constraints. Under these conditions, drivers will generally accept a lower operating speed. Higher design speeds are appropriate in level terrain in which higher design standards can be provided without a major increase in cost. Safety could be jeopardized if high design standards are not provided in flat terrain, because drivers tend to overdrive the road.

In remote areas in which trips are long, it is perceived that drivers tend to drive at higher speeds. Higher design speeds should generally be selected for roads that constitute a long trip. However, there are difficulties in defining a long trip and in identifying the relationship between trip length and design speed. Research is required on this aspect of design speed selection.

Service function also influences the selection of design speed. Roads that serve adjacent developments, with numerous access points, should have a lower design speed. Recreational roads generally have lower design speeds because of environmental or aesthetic considerations, or because of adjacent development.

Although the design speed concept has been used in these standards, its application to the design of low-volume roads is subject to question by many. It is considered unrealistic and uneconomical to attempt to balance all of the physical features of the road to a consistent design speed, particularly in rolling or mountainous terrain. If low-volume roads are to be low-cost roads, they should be designed to fit the terrain and conditions instead of being designed to some preselected design speed. However, until more research is performed on this aspect of low-volume roads, the design speed concept will continue to be used for low-volume roads.

Alignment Elements

The alignment elements developed for low-volume roads are primarily determined from the design speed using the same physical relationships developed for other road classifications. However, some modifications have been made to the RTAC standards developed for roads of higher classifications to satisfy specific requirements for low-volume roads. These modifications relate to vertical curvature, gradients, and the development of superelevation.

Minimum Stopping Sight Distance

Minimum stopping sight distances for low-volume roads are based on a fixed brake reaction time of 2.5 sec and on friction values for wet pavement in poor condition, as for roads of a higher classification. Although friction values for gravel and earth roads have been developed through research, the results have not been translated into usable standards. This is of particular concern because although the friction values for wet pavement in poor condition may reflect some gravel surface conditions, they do not reflect all the variations in surface type and conditions that occur on gravel and earth roads. Until further research is undertaken, wet pavement friction values will continue to be used to establish the minimum stopping site distances on gravel and earth roads.

The minimum stopping sight distance on one-lane, two-way roads is twice that required on two-lane roads based on the assumption that both drivers use the same brake reaction time and are traveling at the same speed. Both AASHTO and the U.S. Forest Service have adopted this standard (*2, 4*).

Horizontal Alignment

Lateral friction factors for gravel roads and earth roads are assumed to be the same as for wet pavements in poor condition. Like the development of stopping site distance, lateral friction values for gravel roads and earth roads have not been translated into usable standards.

A maximum relative gradient for tangent runoff of 1:200 is recommended for superelevating roadways. A value of 1:400 is generally used on roads of a higher classification. However, on low-volume roads in which surface type may be of a lower quality, 1:200 minimizes the length of roadway that has less than the desirable cross-slope for storm water runoff. The AASHTO values for all two-lane highways vary from 1:133 at 30 km/h to 1:222 at 100 km/h (*2*).

The distribution of superelevation rates has been developed for low-volume roads for maximum superelevation rates of 0.08 mm/mm and 0.06 mm/mm, and for normal cross-slopes of 0.02 mm/mm and 0.04 mm/mm. Rates were developed for the 0.04 mm/mm cross-slopes because superelevation is required at a larger radius than when the cross-slope is 0.02 mm/mm for the same design speed.

Vertical Alignment

Crest vertical curvature for stopping sight distance of low-volume roads is based on a fixed object height of 150 mm instead of a fixed tail-light height of 380 mm, which is used for roads of a higher classification. On low-volume roads in which there may be an absence of continuous maintenance, vehicles are more likely to stop for a fixed object, such as logs and washouts, instead of another vehicle. This increases the k-values over that required for roads of a higher classification, as shown in Table 1.

In roads in which maintenance activities are performed on a regular basis, consideration can be given to using k-values developed for the 380-mm object height.

The k-values for one-lane, two-way roads are based on the height of the opposing vehicle, which is assumed to be 1.30 m, because two vehicles approaching each other is the governing condition for minimum stopping sight distance instead of a vehicle approaching a fixed object.

Grades

A review of Canadian road agency standards showed that the maximum gradients used for low-volume roads were similar to the maximum gradients recommended for the RTAC rural local undivided (RLU) road classification. When compared with the AASHTO suggested maximum gradients for local rural roads, the Canadian road agency standards were quite conservative, and because of the low volumes, inappropriate.

The selection of the design maximum gradient depends on many factors including topography, volume of traffic, traffic mix, truck size, and construction costs. An economic analysis should ideally be undertaken to determine the maximum gradient for the design speed and traffic mix.

Until further research is performed on the relationship between gradient and maintenance costs, road user costs, and stopping distance, the suggested maximum gradients will be based on the suggested AASHTO maximum gradients for local rural roads (*2*).

A comparison between recommended gradients for low-volume roads and those of the next highest road classification, RTAC RLU, is shown in Table 2 (*1*).

Cross-Section Elements

Cross-section elements for low-volume roads were developed based on traffic volume, traffic mix, design classification, design speed, and surface type for two-lane earth roads; two-lane gravel roads; two-lane surfaced roads; one-lane, two-way roads; and one-lane, one-way roads.

The cross-section elements for two-lane earth and gravel roads and two-lane surfaced roads are shown in Figures 1 and 2, respectively. The cross-section elements for one-lane, two-way and one-lane, one-way low-volume roads are shown in Figures 3 and 4, respectively.

Roadway Width

The development of roadway widths for two-lane low-volume roads was based on an analysis of roadway widths currently used by Canadian road agencies, and those recommended by AASHTO and NCHRP (*6*). The following assumptions were made in the analysis:

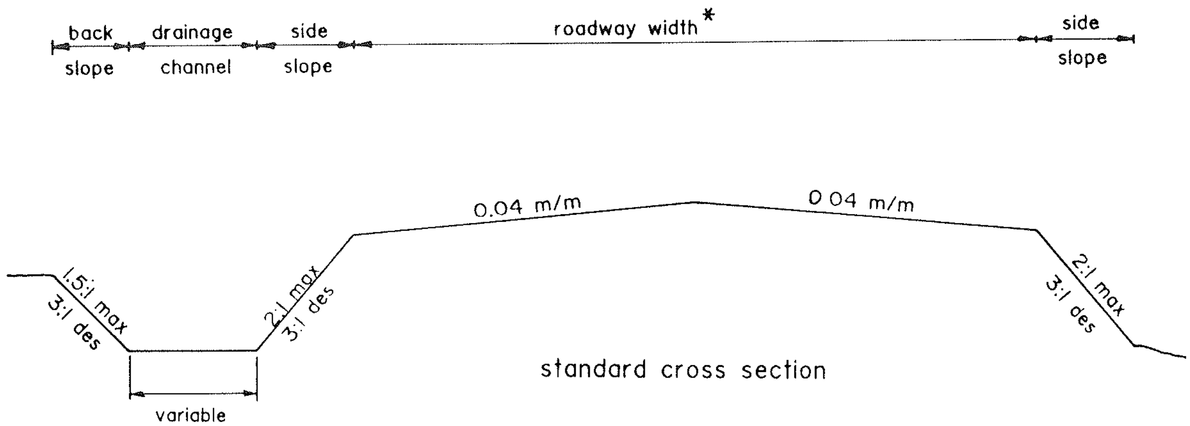
- Some road agencies have roadway width standards for ADTs less than 200 vpd, whereas others have standards for ADTs less than 250 vpd, less than 400 vpd, and 100 to 500 vpd. Roadway widths in these ranges were assumed valid in the analysis.
- Agencies that did not specify ADTs submitted standards for rural local roads, which were also included in the analysis.
- Most agencies do not consider truck volumes in their roadway width standards. Therefore, the roadway widths used by Canadian road agencies were assumed to be applicable to all truck volumes.

TABLE 1 COMPARISON OF K-VALUES FOR LOW-VOLUME ROADS AND HIGHER CLASSIFICATION ROADS

	Crest K-Value (m)	
Design Speed (km/h)	Low-Volume Roads (Object Height = 150 mm)	Higher Classification Roads (Object Height = 380 mm)
30	3	-
40	5	4
50	12	7
60	18	15
70	30	22
80	50	35
90	70	55
100	100	70

TABLE 2 COMPARISON OF GRADIENTS

Design Speed (km/h)	Low-Volume Road (Maximum Gradient %)	RLU (Maximum Gradient %)
30	11-16	7-11
40	11-15	7-11
50	10-14	7-11
60	10-13	7-11
70	9-12	6- 9
80	8-10	6- 8
90	7- 9	5- 7
100	6- 8	5- 7



class LVR (all categories)	roadway width** m: ADT less than 100, trucks less than 15 ADTT	ADT less than 100, trucks greater than 15 ADTT	ADT 100-200, trucks less than 15 ADTT	ADT 100-200, trucks greater than 15 AADTT
100	7.4	7.8	7.4	7.8
90	7.0	7.4	7.4	7.8
80	7.0	7.4	7.0	7.4
70	6.6	7.0	7.0	7.4
60	6.6	7.0	6.6	7.0
50	6.0	6.4	6.2	6.6
40	6.0	6.4	6.2	6.6
30	5.6	6.0	6.0	6.4

* To allow for future gravelling of earth roads, consideration should be given to constructing the initial roadway width to accommodate the gravel thickness.

** Where traffic barrier is used, increase roadway width by 0.5 m on traffic barrier side of roadway. Roadway widths do not include roundings.

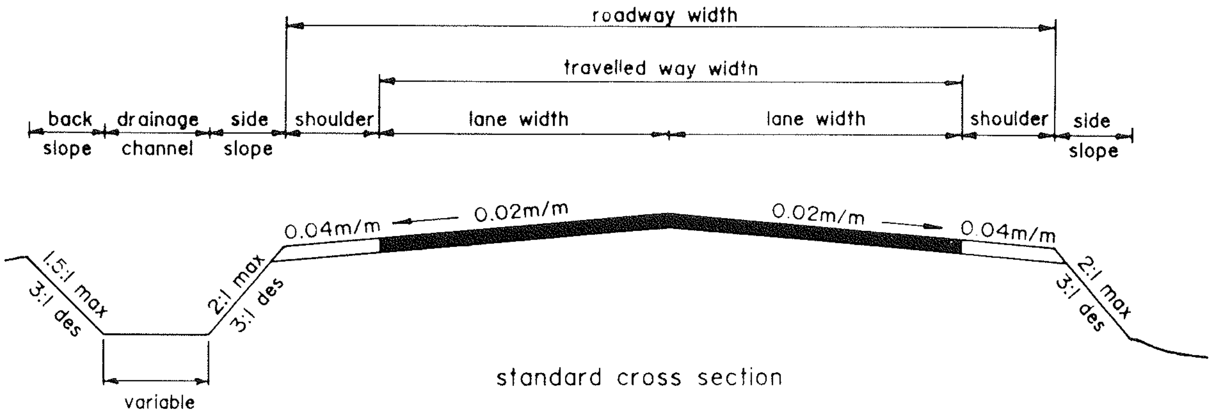
FIGURE 1 Roadway width versus design speeds of various road agencies (ADT <50).

The average roadway widths of the Canadian road agency for gravel roads with ADTs of less than 100 vpd and 100 to 200 vpd are shown in Table 3. Also shown are minimum roadway widths recommended by AASHTO for ADTs less than 250 vpd, and NCHRP for ADTs less than 400 vpd (*2, 6*)

Some agencies further broke the ADT volumes down into 0 to 50 vpd, 50 to 100 vpd, 100 to 150 vpd, and 150 to 200 vpd. However, the differences in roadway widths between 0 to 50 vpd and 50 to 100 vpd and between 100 to 150 vpd and 150 to 200 vpd were negligible. The roadway widths used by Canadian road agencies are shown in Figures 5 to 8. Also shown are the roadway widths recommended by AASHTO, NCHRP, and the National Association of Australian State Road Authorities (NAASRA) for design speeds from 30 km/h to 100 km/h for ADTs of 50 vpd, 50 to 100 vpd, 100 to 150 vpd, and 150 to 200 vpd, respectively.

As can be seen from Table 3, the average of the Canadian road agency road widths is substantially higher than that of AASHTO for all design speeds and that of NCHRP for design speeds of 60 km/h and less (*2, 6*). The NCHRP roadway widths satisfy safety requirements for tracking and lateral clearance and are significantly higher than those of AASHTO and Canadian road agencies for design speeds higher than 60 km/h. However, they apply to ADT volumes up to 400 vpd.

The roadway widths used by Canadian road agencies are greater than those of AASHTO for all design speeds and those of NCHRP for design speeds 60 km/h and less (*2, 6*). This is because many of these agencies provide wider lanes and wider

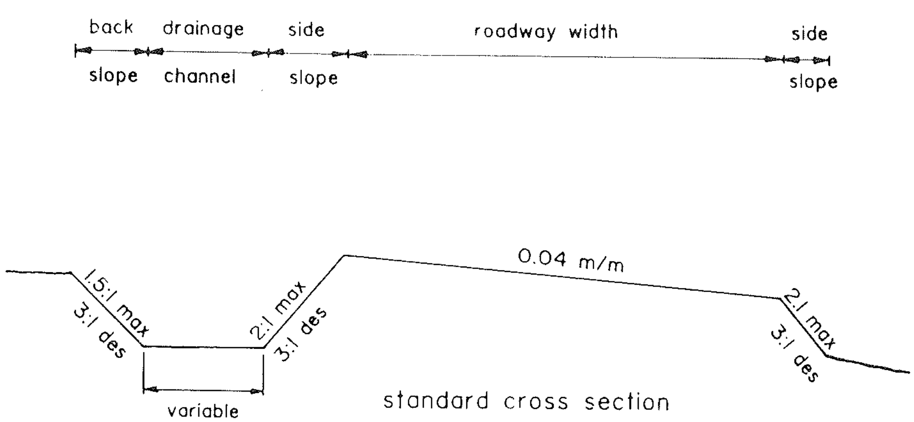


class LVR (all categories)	roadway width** m	travelled way width m	lane width m	shoulder width* m
	ADT less than 200			
100	8.4	7.4	3.7	0.5
90	8.4	7.4	3.7	0.5
80	8.0	7.0	3.5	0.5
70	8.0	7.0	3.5	0.5
60	7.6	6.6	3.3	0.5
50	7.2	6.2	3.1	0.5
40	7.2	6.2	3.1	0.5
30	7.0	6.0	3.0	0.5

* where traffic barrier is used, increase shoulder width by 0.5 m
** roadway widths do not include roundings

Note: Surfaced roads are roads on which the travelled lanes have been physically delineated by some form of bituminous or concrete surface.

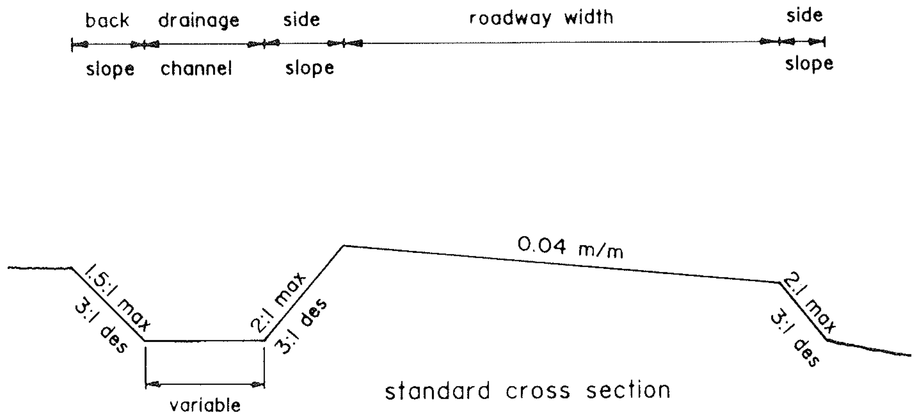
FIGURE 2 Roadway width versus design speeds of various road agencies (ADT 50 to 100).



category	maximum ADT	maximum design speed km/h	roadway width* m
rural road systems and roads to or within isolated communities	50	50	4.0
recreational roads	50	50	4.0
resource development roads	100	50	4.0

* where traffic barrier is used, increase roadway width by 0.5 m on traffic barrier side of roadway.
roadway widths do not include roundings.

FIGURE 3 Roadway width versus design speeds of various road agencies (ADT 100 to 150).



category	maximum ADT	design speed km/h	roadway width* m
recreational roads	200	30-100	5.5

* where traffic barrier is used, increase roadway width by 0.5 m on traffic barrier side of roadway.
roadway widths do not include roundings.

FIGURE 4 Roadway width versus design speeds of various road agencies (ADT 150 to 200).

TABLE 3 ROADWAY WIDTHS FOR TWO-LANE GRAVEL ROADS

	Roadway Width								
	Average of Canadian road agencies		AASHTO	NCHRP Report 214		Recommended roadway widths			
				ADT less than 400		ADT less than 100		ADT 100-200	
design speed	ADT less than 100	ADT greater than 100	ADT less than 250	ADTT less than 14	ADTT greater than 14	ADTT less than 15	ADTT greater than 15	ADTT less than 15	ADTT greater than 15
km/h	m	m	m	m	m	m	m	m	m
100	8.4	9.0	7.3	-	-	7.4	7.8	7.4	7.8
90	8.5	8.6	7.3	-	-	7.0	7.4	7.4	7.8
80	7.8	8.0	7.3	9.1	9.1	7.0	7.4	7.0	7.4
70	7.2	7.4	7.3	7.9	7.9	6.6	7.0	7.0	7.4
60	7.4	7.6	-	6.7	7.3	6.6	7.0	6.6	7.0
50	7.2	7.4	6.7	6.1	6.7	6.0	6.4	6.2	6.6
40	7.7	8.0	6.7	6.1	6.7	6.0	6.4	6.2	6.6
30	6.8	7.1	6.7	5.5	6.1	5.6	6.0	6.0	6.4

Note: Roadway width of gravel roads is the distance between the intersections of the side slopes and the roadway surface.

shoulders and, in many cases, include rounding as part of the roadway width. This additional width is not considered appropriate for low-volume roads because volumes are low, the frequency of traffic conflicts is minimal, and, in practice, drivers tend to travel down the center of the roadway until they meet an oncoming vehicle.

The recommended roadway widths for gravel roads shown in Table 3 do not consider shoulder widths or rounding widths. The roadway widths were developed for two volume categories, for ADT volumes of less than 100 vpd and between 100 and 200 vpd, to reflect the slight increase in roadway widths found in the analysis of Canadian road agency standards in which the ADTs exceeded 100 vpd.

The roadway widths were developed to account for truck traffic, based on the information given in the NCHRP report. They do not provide for emergency or leisure stops because the frequency of traffic conflicts on low-volume roads associated with stopped vehicles does not justify the additional width for sheltering them.

The roadway widths for two-lane surfaced roads include a 0.5-m shoulder adjacent to the traveled way for lateral support of the roadway structure. The recommended traveled way width provides adequate tracking and lateral clearance for all ADT volumes less than 200 vpd and for all truck volumes.

One-lane, two-way roads were introduced for low-volume, low-speed conditions. For rural road systems, roads to or within isolated communities, and recreational roads, one-lane, two-way roads can be used when the ADT is less than 50 vpd and for design speeds of 50 km/h or less. On roads used exclusively for resource development, one-lane, two-way roads can be used when the ADT is less than 100 vpd, and for design speeds of 50 km/h or less. For reasons of safety, one-lane, two-way roads should only be considered when the following conditions can be satisfied:

- Operating speeds are limited to 50 km/h or less,
- The road is short in length,
- The road serves a single purpose,

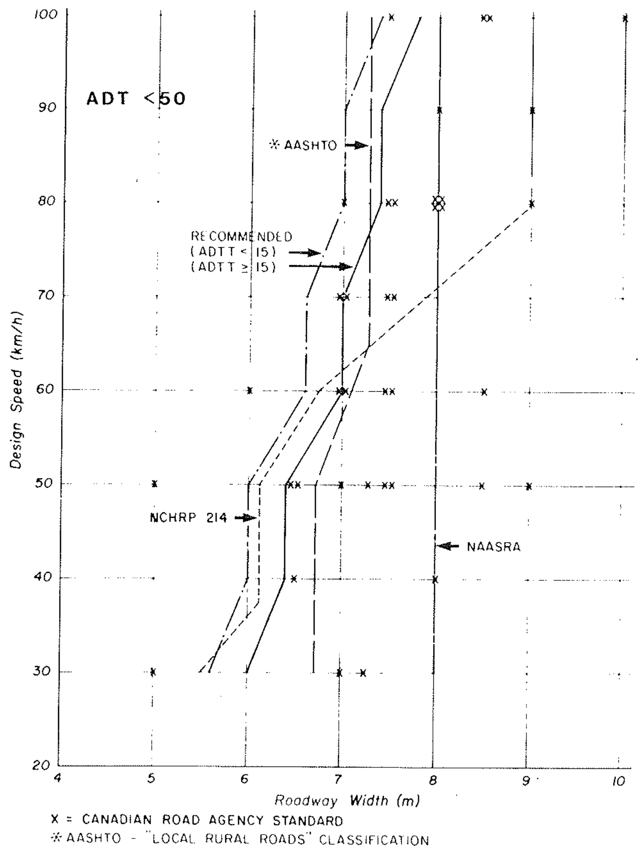


FIGURE 5 Cross-section elements for two-lane, low-volume earth and gravel roads.

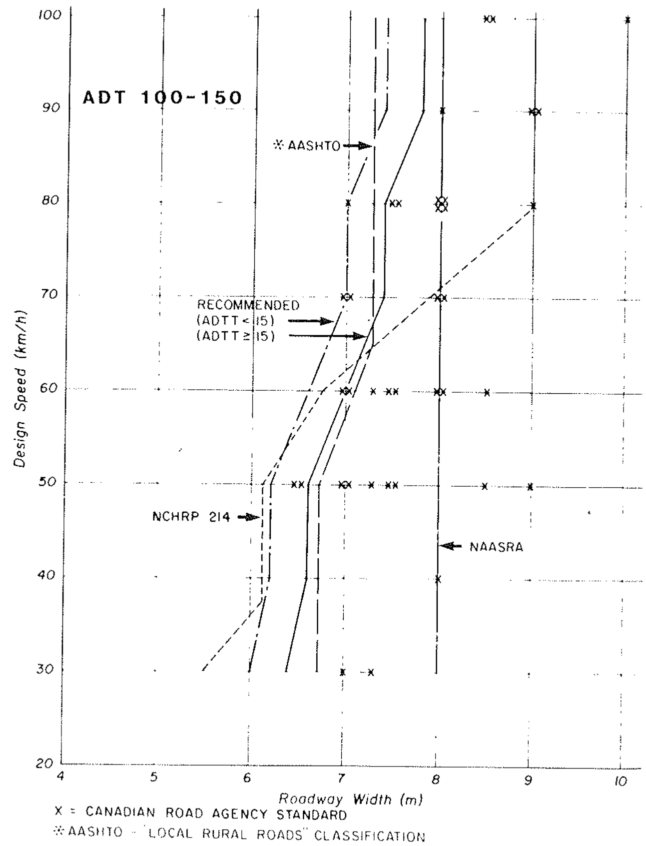


FIGURE 7 Cross-section elements for one-lane, two-way, low-volume roads.

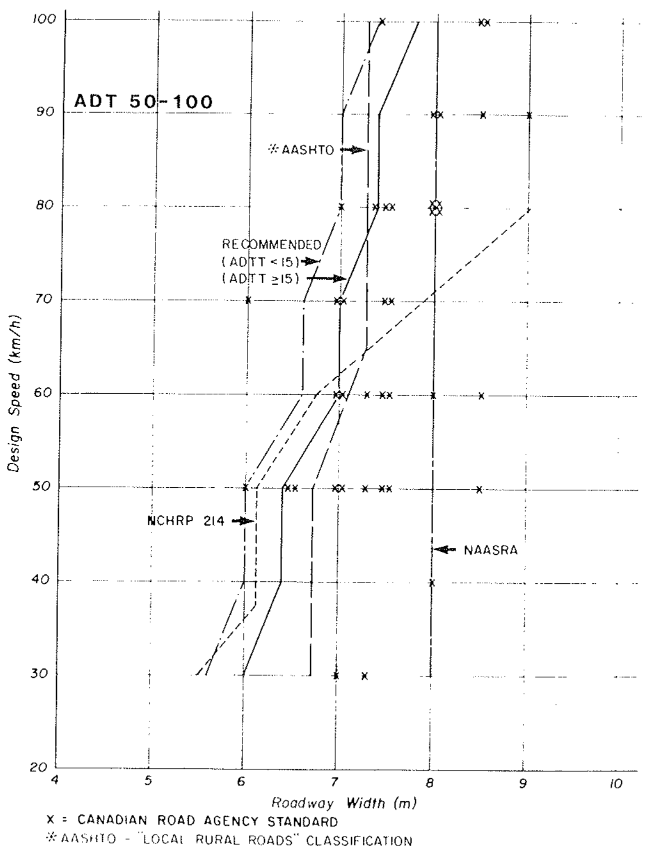


FIGURE 6 Cross-section elements for two-lane, low-volume surfaced roads.

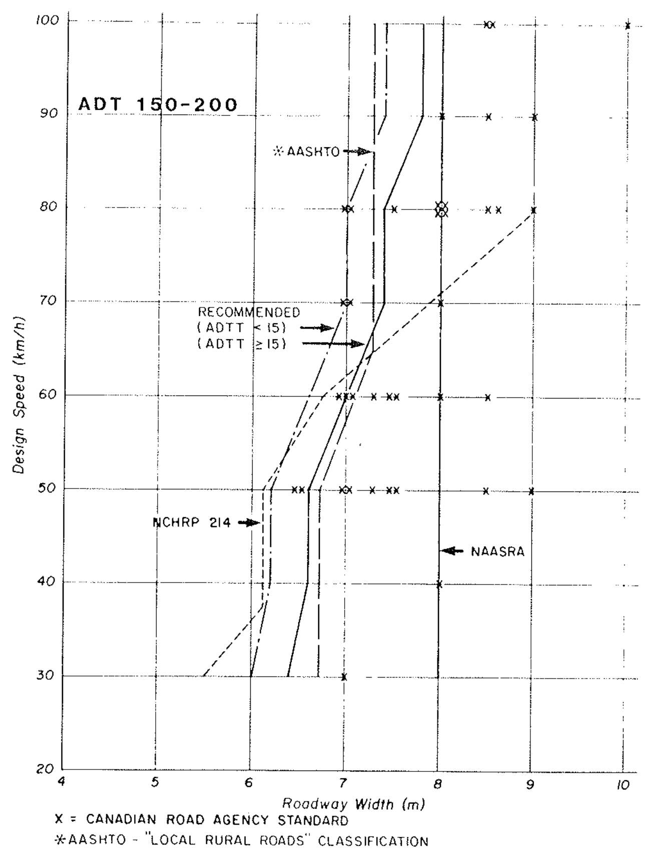


FIGURE 8 Cross-section elements for one-lane, one-way, low-volume roads.

- The road is clearly signed as to its configuration, and
- Turnouts for passing are provided.

Respondents to the questionnaire generally favored the use of one-lane, two-way roads under low-speed, low-volume conditions. AASHTO recommends one-lane, two-way roads for recreational and resource development roads when the ADT is less than 100 vpd and for design speeds of 50 km/h or less (*2*). The U.S. Forest Service also has standards for one-lane, two-way roads (*4*).

The recommended roadway width for one-lane, two-way roads is 4.0 m. The width of 4.0 m prevents the road from being used as a two-lane facility. AASHTO recommends a roadway width for one-lane, two-way roads of 3.7 m or 4.0 m (*2*).

Turnouts must be provided for passing. They should be intervisible with a spacing of approximately 300 m. On roads used exclusively for resource development, turnout spacing can be increased if the vehicles are equipped with two-way radio communication.

The National Association of Australian State Road Authorities (NAASRA) recommends one-lane, two-way roads for AADT volumes less than 150 vpd for all design speeds (*7*). However, the cross-section consists of a 3.5-m sealed lane and 1.5-m to 2.5-m shoulders for a total roadway width of 6.5 m to 8.5 m. Turnouts are not provided, because the total roadway width is ample for passing.

One-lane, one-way roads have been included for use in recreational sites. AASHTO recommends one-way roads in recreational sites and Parks Canada recommends one-way roads for internal and perimeter campground roads (*2, 3*). The recommended roadway width of 5.5 m is based on Parks Canada's recommended cross-section arrangement, which consists of a 4.5-m lane and 0.5-m shoulders to allow other vehicles to pass a stopped vehicle. Standards have been developed for one-lane, one-way roads for all design speeds up to 100 km/h and for an ADT of up to 200 vpd in one direction because head-on conflicts are eliminated.

Other design considerations that affect roadway width are parking, leisure stops, and overwidth trucks. In some cases, low-volume roads may be located in an area in which vehicle parking on the roadway is a requirement. These roads generally have a low design speed and, therefore, a narrow roadway width. Consideration should be given to widening the roadway to accommodate vehicle parking on one side. In such cases, the suggested maximum roadway width is 8.0 m.

Frequent leisure stops may occur in recreational areas such as historic sites or scenic viewpoints. As a safety requirement, consideration should be given to either widening the roadway or constructing turnouts. In such cases, the suggested minimum widening is 3.0 m.

The roadway widths developed for resource development roads meet the safety requirements necessary to accommodate truck widths of 2.6 m. However, in cases in which truck widths greater than 2.6 m are prevalent, it is suggested that the roadway width be increased by the amount the design vehicle width is in excess of 2.6 m for one-lane, two-way roads and by twice this amount for two-lane roads to satisfy safety requirements.

Cross-Slopes

The majority of the respondents to the questionnaire indicated that a cross-slope of 0.04 m/m is preferred on gravel roads to provide effective cross-drainage. AASHTO recommends cross-slopes in the range of 0.02 m/m to 0.06 m/m for earth roads and gravel roads, and 0.015 m/m to 0.030 m/m for surfaced roads (*2*). Cross-slopes of 0.04 m/m are recommended for earth roads and gravel roads, and 0.02 m/m to 0.04 m/m for surfaced roads.

Side-Slopes and Back-Slopes

Maximum earth side-slopes of 2:1 are suggested, depending on the stability of local soils. In mountainous terrain, maximum side-slopes of 1.5:1 may be appropriate in high fill areas to minimize costs. Side-slopes of 3:1 are recommended in the interest of safety. Side-slopes of 2:1 and 3:1 are commonly used by Canadian road agencies. If it is economically feasible, flatter side-slopes should be used.

Maximum back-slopes of 1.5:1 are suggested for low-volume roads, depending on the stability of local soils. Back-slopes of 3:1 are recommended in the interest of safety. Some Canadian road agencies use 1.5:1 back-slopes.

For local rural roads, AASHTO states that side-slopes should not be steeper than 2:1 in cut sections, and back-slopes should not exceed the maximum required for stability.

FUTURE RESEARCH

During the development phase of the standards, several topics were identified by the Project Steering Committee as requiring further research either because research was lacking on the subject, or because available research was only applicable to roads of a higher classification. Areas that require future research are described in the following sections.

Friction Factors

As previously stated, stopping sight distance and circular curve radii calculations have been based on friction factors applicable to paved surfaces. Because many low-volume roads have surfaces that consist of earth or granular material, they may require the use of different stopping distances and circular curve radii to account for the different friction factors that can be expected. The friction factors can vary substantially, particularly on granular surface roads. Friction factors would have to be determined for loose gravel and compacted gravel under both wet and dry conditions. Other factors that would affect the friction values are the gravel gradation and maintenance practices, and these would have to be considered in the research.

Review of Maximum Grades and Superelevation Rates

Factors that govern the determination of maximum grades and rates of superelevation are friction of the road surface, surface type, vehicle characteristics and performance, and the desired level of service. Although these factors have been determined experimentally, they have not been sufficiently translated for use in determining Canadian geometric design standards. Maximum design grades for various classes of roads and vehicle type, and desirable superelevation rates in various climatic conditions should be determined to develop Canada-wide standards for these two design elements.

One-Lane, Two-Way Roads

Research on one-lane, two-way roads is required to determine their cost- and safety-effectiveness as opposed to two-lane roads. One-lane, two-way roads have been widely used in European and Scandinavian countries, but their use is limited in Canada and the United States. Under certain traffic volume and road use situations, they could provide an economic alternative to two-lane roads. There is, however, a need to develop more information on their operation, including appropriate traffic volume levels and accident potential, to support and expand their use.

Developing Optimum Widths for Structures on Low-Volume Roads

The widths and clearances for bridge structures shown in the current RTAC *Manual of Geometric Design Standards for Canadian Roads* may not be appropriate for low-volume roads (*1*). Reduction of bridge widths and clearances may be possible without adversely affecting operation or safety. Optimizing widths for structures on low-volume roads would balance cost savings against the safety and operational requirements. The feasibility of one-lane bridges and the requirements to accommodate farm machinery in agricultural areas would be part of the process.

Safety, Performance, and Costs of Low-Volume Roads

Data are lacking to adequately assess the safety and performance of low-volume roads. Research should include the collection and evaluation of operating, maintenance, and construction cost data from road agencies across Canada as they relate to design speed, road width, and surface type.

Accident data, including the cost of accidents, should also be collected to determine accident rates on low-volume roads and to pinpoint the major cause of the accidents as they relate to horizontal and vertical curvature, sight distance, grades, road width, and surface type.

Economic Analysis Program

A Canadian methodology for the economic analysis of low-volume road projects is required. The methodology would assist road agencies in developing the most economic roadway that satisfies both the road agency and road user requirements. Factors that should be included in the methodology are capital costs, maintenance costs, design life, vehicle operating and travel time costs, and accident costs as they relate to the geometric design elements and road surface type.

Design Speed Related to Trip Length

Although it has not been substantiated, many believe that trip length is a pertinent consideration in selecting design speed. The longer the trip, the greater the desire to travel at higher speeds. The selection of low design speeds for a substantial length of road in flat topography may create unsafe driving conditions because drivers may become impatient and travel at excessive speeds. Conversely, selecting high design speeds for a short length of road may prove to be an uneconomical design. Research is required to establish the relationship between trip length and design speed so that roads may be designed to a safe and economical standard.

Assessment of Design Speed Concepts and Development of an Alternative Approach

The use of the traditional design speed approach may not be appropriate for low-volume roads. It is unrealistic and uneconomical to attempt to balance all of the physical features of a low-volume road to a consistent design speed, particularly in rolling or mountainous terrain. Research should be performed to develop an alternative approach to the design speed concept, such as designing the low-volume road to fit the terrain and estimated desired speed of travel.

Review Volume Level for ADT

The selection of 200 ADTs to define low-volume roads was based on the perception that the majority of lower-volume roads in Canada have traffic volumes less than 200 vpd. Other road agencies classify roads with less than 250, 400, or as high as 1,500 vpd as low-volume. The next volume category for which design standards have been developed in the RTAC manual is less than 1,000 vpd (*1*). Additional research is required to determine if the present design standards for low-volume roads satisfy safety requirements for volumes between 200 and 1,000, or if an intermediate set of standards is required.

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The Development of Low-Volume Roads in India

M. P. DHIR, N. B. LAL, AND K. MITAL

India faces the enormous task of providing all-weather road access to all of its nearly 600,000 villages. The Central Road Research Institute (CRRI) has been engaged in research work on a variety of aspects, some of which are discrete items. Reported in this paper are findings in regard to the study of the socioeconomic impact of road development in rural regions, network planning, pavement design, and efforts to evolve intermediate technologies. The study of socioeconomic aspects indicates that some parameters are more directly affected by road development, such as literacy, proportion of non-agricultural workers, unit agricultural yield, and unit fertilizer consumption. Other socioeconomic parameters on which road development has a more indirect effect are facilities for health, education, banking, and postal services. A new planning methodology that is based on graph theory is presented. In its present form, it provides a simple method for more rational decision-making, but it has certain limitations. Reported herein are findings from comprehensive work performed by CRRI on the design of pavement for low-volume roads to allow for such elements as traffic of solid-wheeled carts, varying subgrade moisture conditions, and minimum acceptable serviceability levels. Intermediate technologies that center around the use of agricultural tractors with agricultural implements and other towed equipment are also mentioned.

Most of India's population continues to live in about 600,000 villages that are scattered in various rural regions. From the standpoint of overall socioeconomic development, it is very important that the transportation network of the country reach out to these numerous population centers. Roads and road transport are preeminently suited to meet the transportation needs involved. A little over one-third of the villages have already been provided with all-weather road access.

Completion of this task calls for great financial outlays and, therefore, utmost thought is to be given to possible economies. The Central Road Research Institute (CRRI) has been devoting considerable effort toward the development of improved and more cost-effective techniques for the planning, design, construction and maintenance of low-volume roads. The CRRI had done work earlier on the approach to planning rural road networks that applied the concept of minimal spanning trees. The CRRI recently had the opportunity to review nine district-level studies that were undertaken in different parts of the country to assess the socioeconomic impact of road development in rural regions. The review was undertaken as a step toward further analysis, synthesis, and possible rationalization.

The development of techniques for the use of soils and other local materials in road construction has been a major concern. A number of small projects were arranged in different parts of the country in the form of test tracks and demonstration works. The feedback from these and other projects provided the basis for such aspects as the suitability of techniques for benefication of local materials available in different parts of the country, estimation of the highest subgrade moisture content, estimation of subgrade strength from index properties, deterioration of low-volume roads, and relationships to pavement design of different serviceability levels.

Road construction in India continues to have a high level of manual input. The CRRI has been working on the development of intermediate technologies appropriate to the small and scattered works of village roads. This work has consisted of technologies based on the use of agricultural implements, animal power, and the agricultural tractor as a prime mover. The CRRI has also developed new pavement systems for low-volume roads in different regions with certain special conditions.

A number of special programs have been launched from time to time to speed the socioeconomic development of rural communities. The construction of low-volume roads is an important component of these programs. The organizational structure has also been under review. A number of exercises have been undertaken in recent years in government circles and in the Indian Roads Congress to update the management of low-volume roads.

THE DEVELOPMENT OF ACCESS ROADS

When India launched its 5-year development plan in 1951, the total road length was about 400,000 km, nearly two-thirds of which was unsurfaced. Road development has been a notable component, both directly and indirectly, in the various 5-year plans that have launched since then. A number of programs were created with such objectives as area development, provision of minimum needs, and generation of employment that have contributed notably to the construction of low-volume roads.

The total number of villages in India are broken down according to population size and the percentage of villages that are connected by all-weather roads as of April 1, 1986, in the following table.

Population	*Total Number of Villages*	*Percent Connected With Roads*
1,500 and above	69,408	74
1,000 to 1,500	56,609	54
Less than 1,000	466,076	29
Total	592,093	37

It can be seen that a great majority of the villages have a population of less than 1,000. The percentage of the total number of villages that are connected by roads is 37. In order to connect all villages with roads, the length of the low-volume road network would have to be increased to 2.2 million km (Figure 1) (*1*).

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SOCIOECONOMIC ASPECTS

It is generally accepted that the provision of roads contributes notably to a variety of socioeconomic activities. If this contribution could be quantified, an assessment could be made of the priority to be placed on road development in an economy of highly competing needs. Some attempts have been made in the past to understand the interactive relationship between roads and development. Studies with similar objectives were launched under the aegis of the Indian Roads Congress in nine districts (Figure 2) that were typical in regard to such factors as the level of development already reached, the potential for development, the nature of the economic base, and the characteristics of the terrain. These studies were entrusted to separate agencies. It was soon realized that the data and findings from individual district studies would have to be further explored to place data on as uniform a base as possible, and to rationalize the differences and findings. This task was entrusted to the CRRI and some findings can be reported based on work performed thus far.

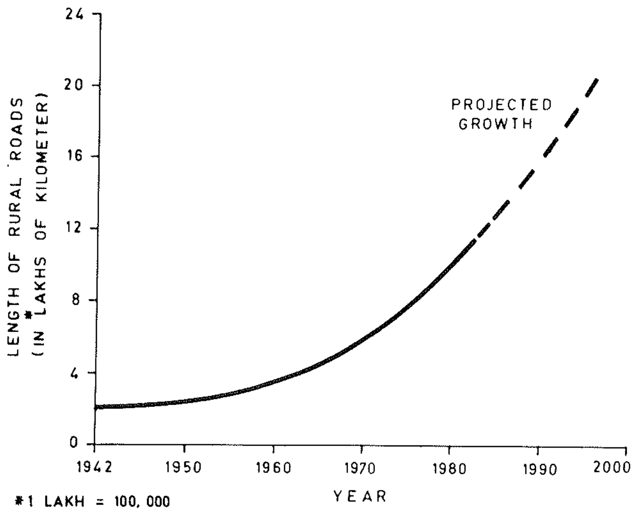


FIGURE 1 Growth of rural roads in India.

FIGURE 2 Locations of district-level studies on socioeconomic aspects of rural roads.

It can be anticipated that the existence of a road in the neighborhood of a rural community can contribute to development directly and indirectly. The effects of the availability of road access on socioeconomic activities frequently depend on the availability of other inputs. For example, the availability of road transport can contribute to the development of agriculture only if irrigation facilities, improved seeds, fertilizers, and bank loans are available. The nine district-level studies were performed by seven different agencies. Although these agencies were given common terms of reference, there have been variations in detail in regard to the collection and treatment of data.

When CRRI undertook the task of rationalization and synthesis, it was soon realized that a district with its own heterogeneity is too large a unit for the impact to be assessed. The districts were therefore divided into subdistricts according to terrain (plain, rolling, or hilly) and level of development (relatively undeveloped and relatively developed). Two districts also had a sizable tribal population.

The three road parameters studied were road length/unit area, road length/unit population, and level of road access, or distance between a village and the nearest road. Presented in Figures 3 to 10 are some of the plots that indicate the effect of the intensity of road development on certain socioeconomic parameters. These data are in regard to the subdistricts with plain or rolling terrain, but include both developed and undeveloped pockets. The following socioeconomic parameters are presented:

- Literacy level, or percent literate (Figure 3),
- Nonagricultural workers as a percentage of total workers (Figure 4),
- Agricultural yield in tonnes/hectare (Figure 5),
- Fertilizer consumption in tonnes/1000 hectares (Figure 6),
- Cooperative banks/100,000 persons (Figure 7),
- Primary schools/1,000 persons (Figure 8),
- Primary health care centers/100,000 persons (Figure 9), and
- Post offices/100,000 persons (Figure 10).

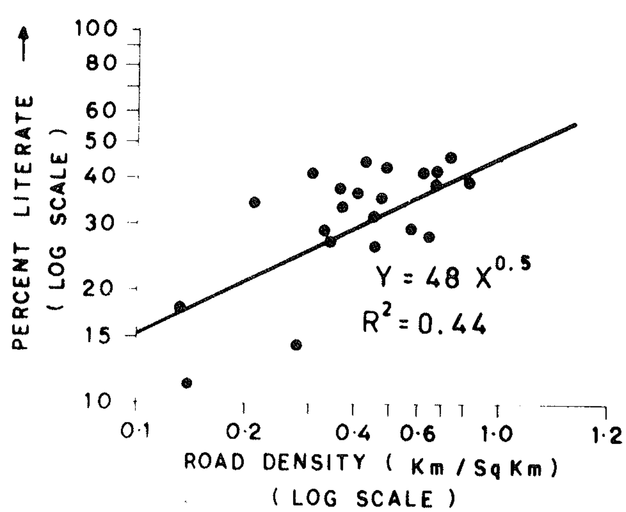


FIGURE 3 Percent literate versus road density.

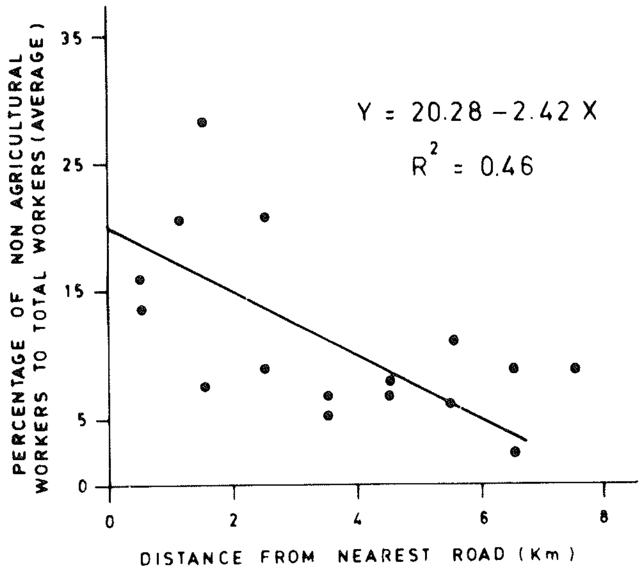


FIGURE 4 Nonagricultural workers versus distance from nearest road.

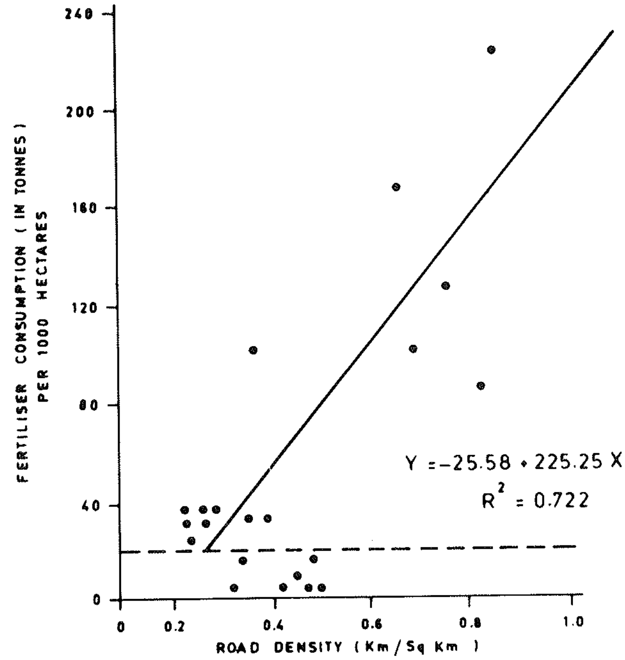


FIGURE 6 Fertilizer consumption versus road density.

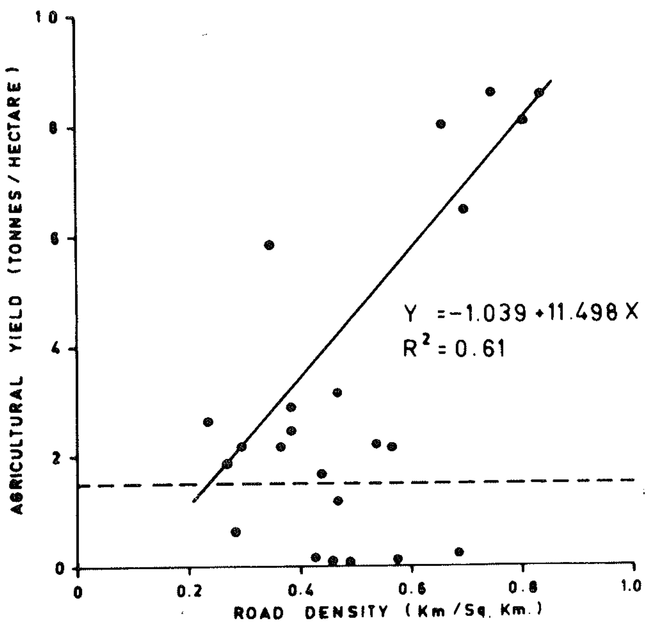


FIGURE 5 Agricultural yield versus road density.

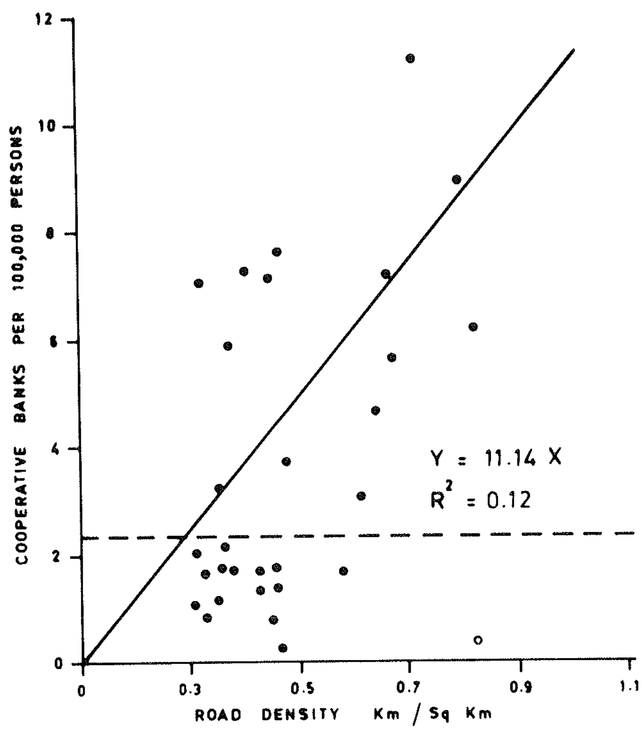


FIGURE 7 Cooperative banks versus road density.

It can be observed from the plots in these figures that there are pockets in which certain parameters had not been duly influenced by road development. It is believed that the function of the road could not be exploited in these cases for want of other needed inputs. The correlation and other coefficients were arrived at after excluding these pockets, as shown in Figures 5 to 9. The same also applies to the stray plots that deviated greatly from the general trend for various reasons.

The plots presented in Figures 3 to 10 can be divided into the following two groups: aspects in which the function of the road is more direct (Figures 3 to 6), and aspects in which the function of the road is indirect (Figures 7 to 10).

Direct Aspects

The literacy rate significantly increases with road density (Figure 3) according to the following relationship:

$$\text{Percent literate} = 48 \times (\text{road density in km/km}^2)^{0.5}$$
$$(R^2 = 0.44)$$

It can be surmised that the provision of roads enables students to travel to more distant schools. However, it can also be

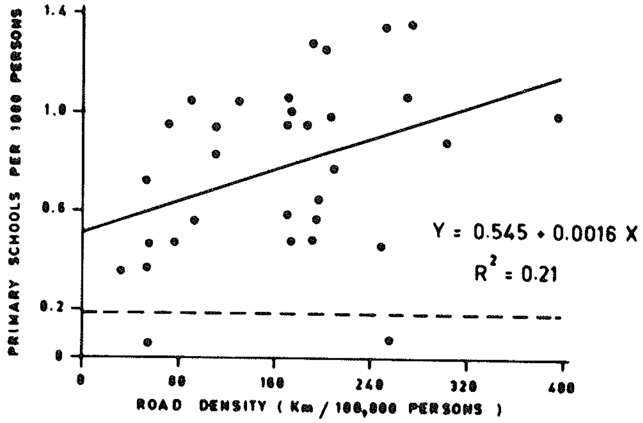


FIGURE 8 Primary schools versus road density.

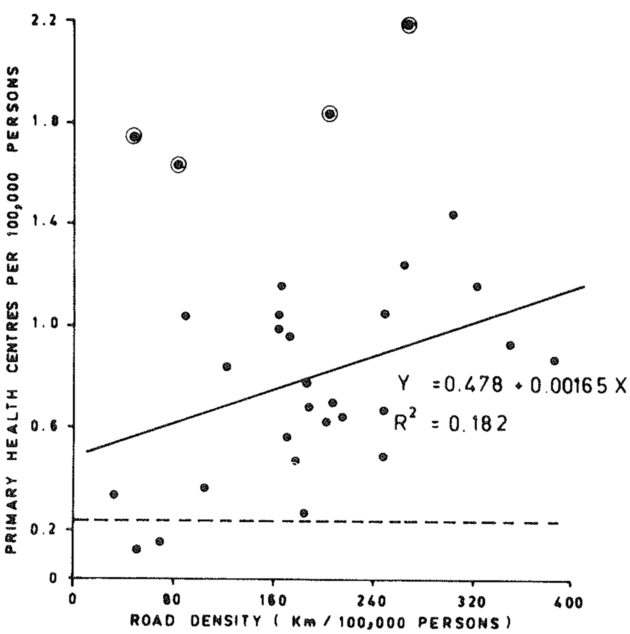


FIGURE 9 Primary health care centers versus road density.

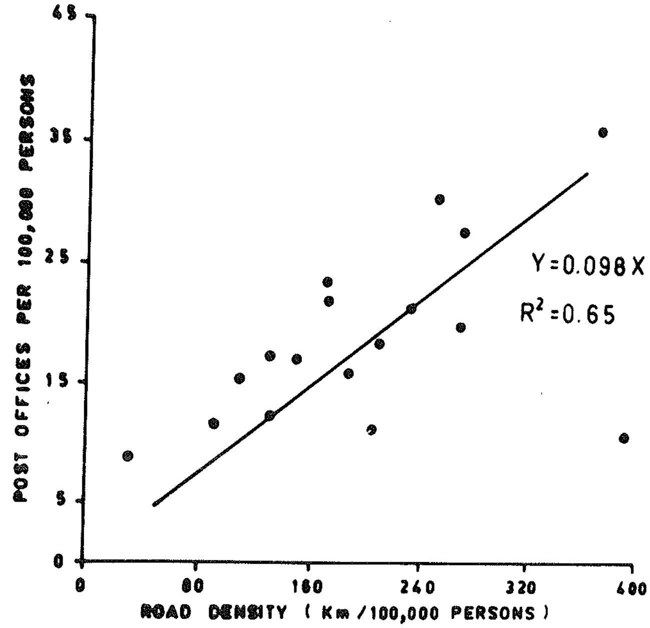


FIGURE 10 Post offices versus road density.

surmised that the availability of teachers improves because they can travel to rural regions that were previously inaccessible.

The effect of roads on the extent to which the populace depends on agriculture is demonstrated in Figure 4. It can be seen that the percentage of nonagricultural workers is very low in pockets that are distant from roads, but that this percentage increases as road access improves according to the following relationship:

Percentage of nonagricultural workers to total workers = 20.28 - 2.42 (distance from nearest road in km)
(R^2 = 0.46)

This can be taken to mean that opportunities for employment and development of skills in nonagricultural activities improve with road access.

After the pockets with little or no road development are eliminated, it can be seen (Figure 5) that a fairly linear relationship exists between unit agricultural yield and road density, as follows:

Agricultural yield in tonnes/hectare = 11.498 (road density in km/km^2) - 1.039
(R^2 = 0.61)

This can be taken to mean that road access enables a variety of contributions to be made to agricultural yield, such as an improvement in extension work to improve agricultural practices; better availability of inputs like improved seeds and fertilizers; better farmgate prices, which promote intensive agriculture; and other agricultural services.

Proof that road access improves the availability of farm inputs can be found in Figure 6, in which the unit consumption of fertilizer is related to an increase in road density, as follows:

Fertilizer consumption in tonnes/1000 hectares = 225.25 (road density in km/km^2) - 25.58
(R^2 = 0.722)

Fertilizer consumption increases significantly with an increase in road density.

Indirect Aspects

It can be seen in Figure 7 that a linear relationship exists between the availability of banking services and road density, as follows:

Cooperative banks per 100,000 persons
= 11.14 (road density in km/km^2)
(R^2 = 0.12)

It can be surmised that a greater demand for banking services exists, and banks find it easier to operate in rural regions, when roads are available.

Similar, though somewhat subdued, is the correlation obtained between the availability of primary schools and road density (Figure 8), as follows:

Primary schools/1,000 persons
= 0.0016 (road density in km/100,000 persons) + 0.545
(R^2 = 0.21)

The demand for primary education and the availability of teachers in primary schools can be taken to improve with the availability of road access. The decline in the sharpness of correlation trends may be a result of the fact that many villages tend to have their own primary schools. Similar trends are shown in Figures 9 and 10 of the effect of road density on the availability of primary health care centers and post offices. The relationships that emerge are as follows:

Primary health care centers/100,000 persons
= 0.00165 (road density in km/100,000 persons) + 0.478
(R^2 = 0.182)

Post offices/100,000 persons
= 0.098 (road density in km/100,000 persons)
(R^2 = 0.65)

The CRRI has undertaken the collection of further data in three selected subdistricts. Further analyses will be undertaken on the availability of supplementary data. In that regard, the above findings and correlations are not final.

PLANNING OF LOW-VOLUME ROADS

The planning of roads in rural regions can be said to center on the maximized socioeconomic benefit per unit of investment and the attainment of high connectivity. The element of distributive justice also exists. The decision to provide certain villages with road access is currently made on the basis of their population sizes; the linking modes are decided more or less subjectively. In order to prepare master plans at this stage of development, CRRI devised a simple methodology for more rational decision-making. In this methodology the flow of traffic in a rural road network can be considered analogous to the flow of electricity in a circuit (*2*). The road transportation system can be visualized with the following components:

- Market centers (high charge points) that are inter-connected to form a grid that carries relatively high traffic volumes (high charge), and
- Villages of different sizes (that carry a charge propor-tionate to their population) that interact between themselves and with market centers.

High charges at market centers and along main roads attract the smaller charges that are situated at various villages around them. Village roads are assigned weightage in direct proportion to the force of attraction exerted and in inverse proportion to the connecting road length. By using these weightages, the final optimized rural road network can be generated by way of the concept of minimal spanning trees. The rural road network emerges as several minimal spanning trees in which the roots are situated either on main roads or directly at the market centers. The various steps involved in generating the rural road network are described in the following paragraphs and shown in Figures 11*a* and 11*b*.

In the first step, a plan is prepared that shows the location of various villages, market centers, and main roads.

In the second step, the market centers, main roads, and villages of various population levels are assigned the following magnitudes of electric charge:

- Individual village—population is divided by 100 units subject to a maximum of 100,
- Market centers—100 units, and
- Main roads—80 units.

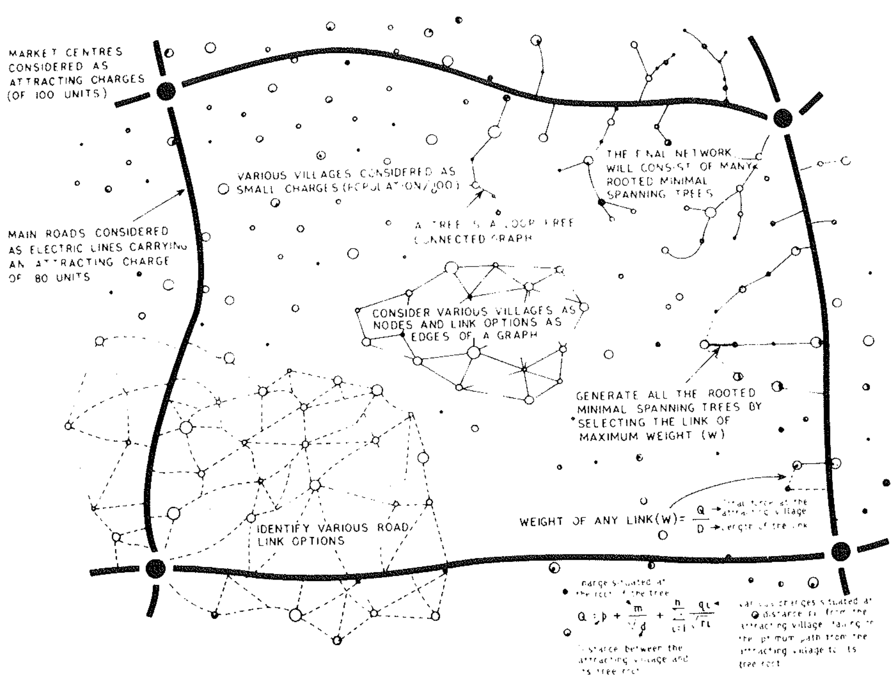


FIGURE 11a **A systems approach to rural road network development.**

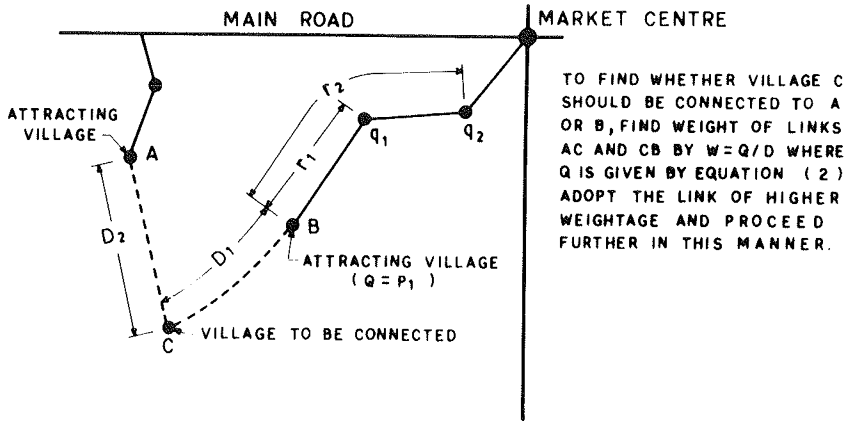


FIGURE 11b Procedure for determining the road links.

In the third step, the villages closest to the market centers and main roads are connected, proceeding toward the interior, depending on the weight (W) of the road length. This weight is given by the following equation:

$$W = Q/D \tag{1}$$

where

D = length of the link option, and
Q = total traffic-attracting force situated at the market center, main road, or village already connected, as the case may be.

The variable Q is equal to 100 or 80 if the village is to be directly connected to the market center or main road, respectively. When a village under consideration is to be connected to a village already connected by a rooted tree, this village is called an attracting village. The variable Q is calculated from the following equation:

$$Q = p + \frac{m}{\sqrt{d}} + \sum_{i=1}^{n} \frac{q_i}{\sqrt{r_i}} \tag{2}$$

where

p = self-charge of the attracting village,
m = charge situated at the tree root (market center or main road),
d = distance of the attracting village from the tree root, and
q_i = charge (population divided by 100) of the i^{th} village that is already planned to be connected, situated at distance r_i from the attracting village.

This procedure is depicted in Figure 11b.

The generation of various rooted, minimal spanning trees will start from their roots. In order to identify the various tree roots, the villages that are closest to the market centers and main roads are connected by selecting the links of maximum weight (W) calculated from Equation 1. All possible roots can be identified in this manner. The various minimal spanning trees are generated side by side by adding links of maximum weight (W) and connecting village to village. The procedure is continued until all villages are connected.

The system thus developed is an idealized one in which the existing rural roads are not considered. In order to make use of the existing road network to the maximum extent possible, the generated network is superimposed on the existing one. Modifications are then made in the planned network by considering the existing roads to have maximum weight. Another refinement is required in cases in which a proposed road link must cross a wide waterway or ridge, which involves a great expenditure. In such cases, it is necessary to examine the proposed links in light of field conditions, and make necessary modifications and adjustments to keep the cost as low as possible. This is similar to cases in which the link options have marginal differences in weightage.

Because the different stretches of a village road that form the branches of a minimal spanning tree have different traffic volumes and composition, it is necessary to classify village roads and sections to determine standards of design and construction. Three different types of village roads are proposed, as follows:

- Type A—village roads that form the main stem of minimal spanning trees and connect a large number of villages, either directly to the market center or through the main road. Such roads should be designed according to CRRI design curves for Category I rural roads, which are discussed in a later section.
- Type B—village roads that form auxiliary branches of minimal spanning trees. These roads could be designed according to CRRI design curves for Category II rural roads.
- Type C—village roads that interconnect the smaller isolated villages. These roads could be designed similar to Type B or developed as earth roads in the first stage.

PAVEMENT DETERIORATION UNDER RURAL TRAFFIC

The rural traffic in India is generally a mix of animal-drawn carts (many of them solid-wheeled), light, pneumatic-tired vehicles such as tractor-trolleys and other intermediate vehicles, and a few heavy vehicles such as buses and trucks. The solid-wheeled, animal-drawn carts impose low wheel loads but very high contact stresses (Figure 12). The repeated operation of these solid-wheeled carts causes deep rutting in the pavement (Figure 13) (*3*). Studies were performed at the Central Road

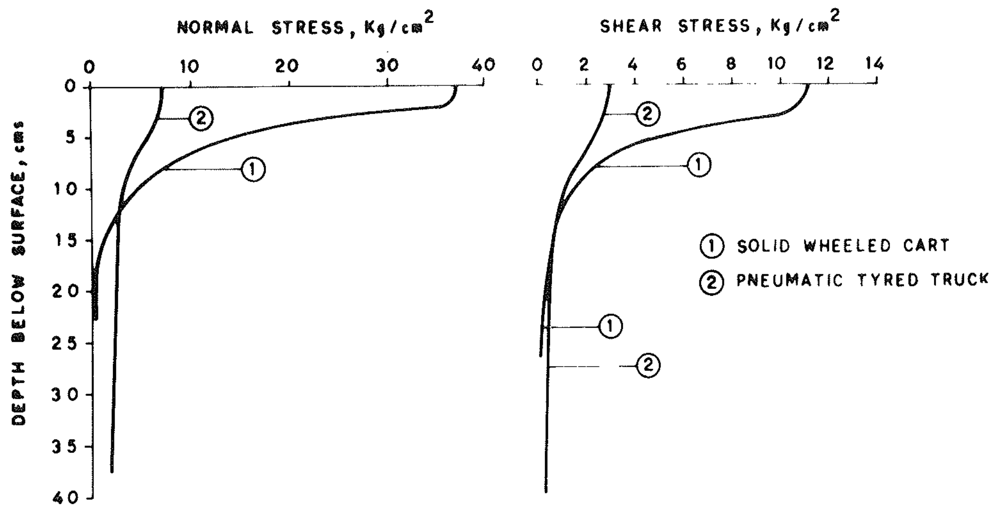


FIGURE 12 Typical variations of normal and shear stress with depth.

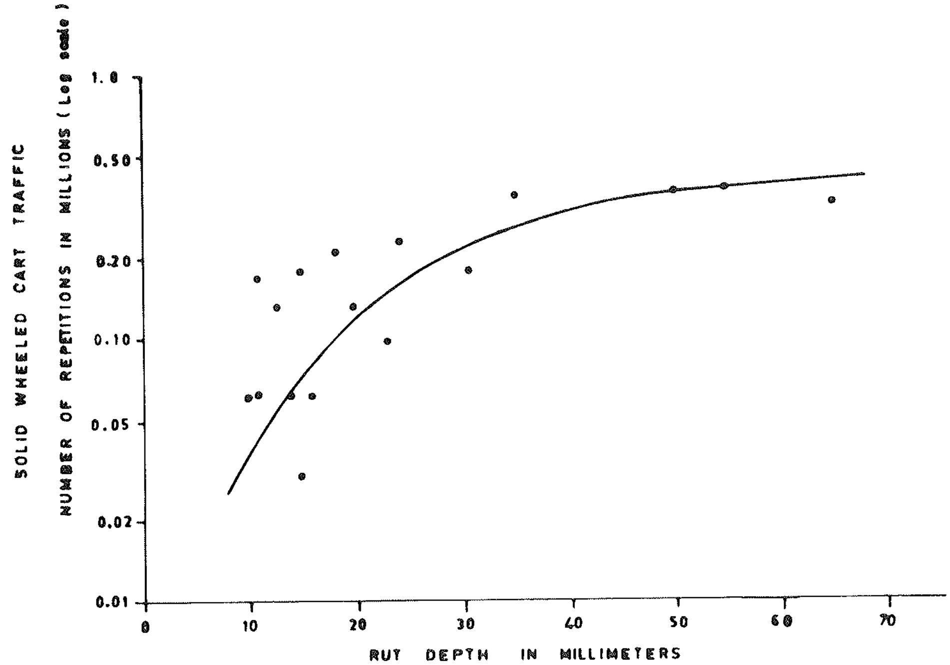


FIGURE 13 Relationship between depth of rutting and repetitions of solid-wheeled cart.

Research Institute on the relative effects of damage caused by different types of rural vehicles (*4*). These studies indicated that solid-wheeled carts caused twice as much damage as the heavy, pneumatic-tired commercial vehicles and five times as much damage as the light, pneumatic-tired vehicles (Figure 14). It was therefore considered appropriate to adopt a composite traffic index that incorporated all three types of vehicles in proportion to the relative effects of their damage for purposes of pavement design. The CRRI is working on field trials of a hub insert developed to improve the contact between the solid wheel and the traveled surface.

In order to control and regulate the evenness of pavement layers and road surfaces, the CRRI has developed a range of devices (*5*). One such device is an unevenness indicator, which essentially is a traveling straight edge with datum wheels fixed at 3 m and a probing wheel in the middle. As the unit is moved at walking speed, it indicates the magnitude of bumps and depressions. Then, according to preset tolerance limits, it marks the nonconforming high and low spots by throwing paint on the surface and sounding a buzzer. The device is shown in Figure 15.

Another device is a profilograph, which consists of a rectangular steel frame supported on a wheel base of 3 m with a central probing wheel. The relative movement of the probing wheel is plotted on a paper recorder, which produces a continuous record of the profile in regard to the moving datum. The device is shown in Figure 16.

PAVEMENT DESIGN CURVES

The low-volume rural roads are short in length. The year-round passability on these roads is more important than the speed of travel. The CRRI developed a set of pavement design curves for

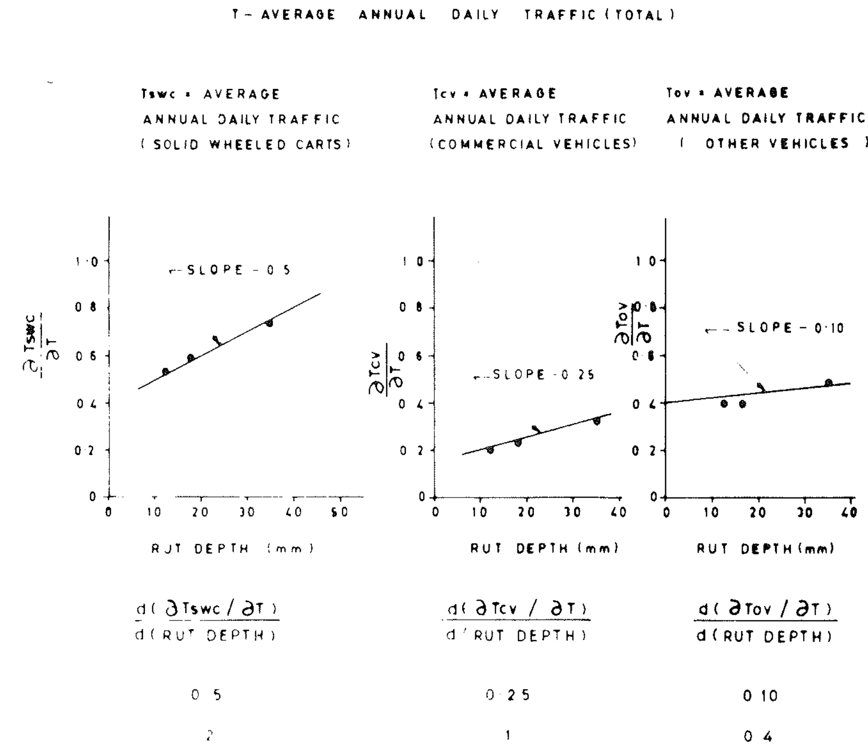


FIGURE 14 Evaluation of the relative damaging effects of solid-wheeled carts, commercial vehicles, and other vehicles on rural roads.

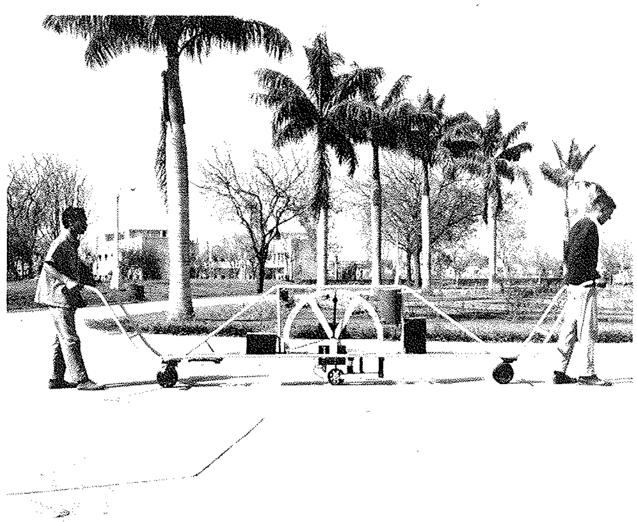


FIGURE 15 Unevenness indicator.

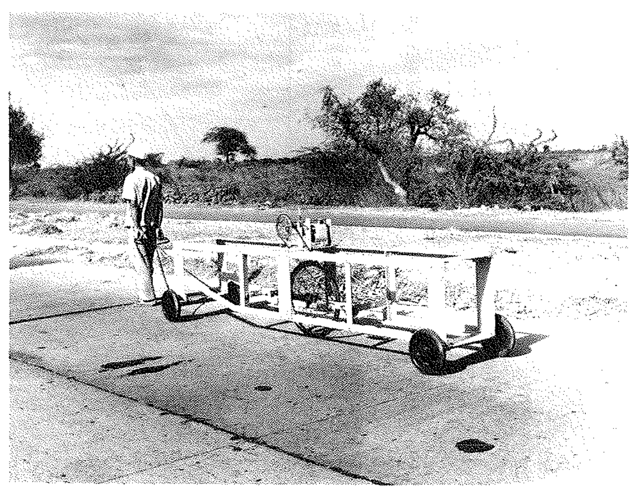


FIGURE 16 Profilograph.

these roads according to two appropriate levels of serviceability. The design moisture content needed to evaluate the subgrade strength greatly influences pavement thickness requirements. A comprehensive survey of a large number of existing roads in different parts of the country was therefore undertaken to develop a simple method of estimating the critical subgrade moisture content. Based on the analysis of data obtained from this survey, the following statistical relationship was developed (*6*).

$$\text{Critical moisture content} = 0.023X_1 + 0.011X_2 + 0.045X_3 + 0.31X_4 + 10.70X_5 + 3.37X_6 + 0.02X_7 - 4.76$$

where

X_1 = percent retained on ISS 2.36-mm sieve,
X_2 = percent fraction passing ISS 2.36-mm sieve and retained on ISS 75 micron,
X_3 = percent fraction passing ISS 75 micron,
X_4 = plasticity index,
X_5 = 1/in situ dry density in gm/cc,
X_6 = 1/shallowest water table (m), and
X_7 = average annual rainfall in cms.

A ready-to-use nomograph that can be used to estimate critical subgrade moisture content is given in Figure 17.

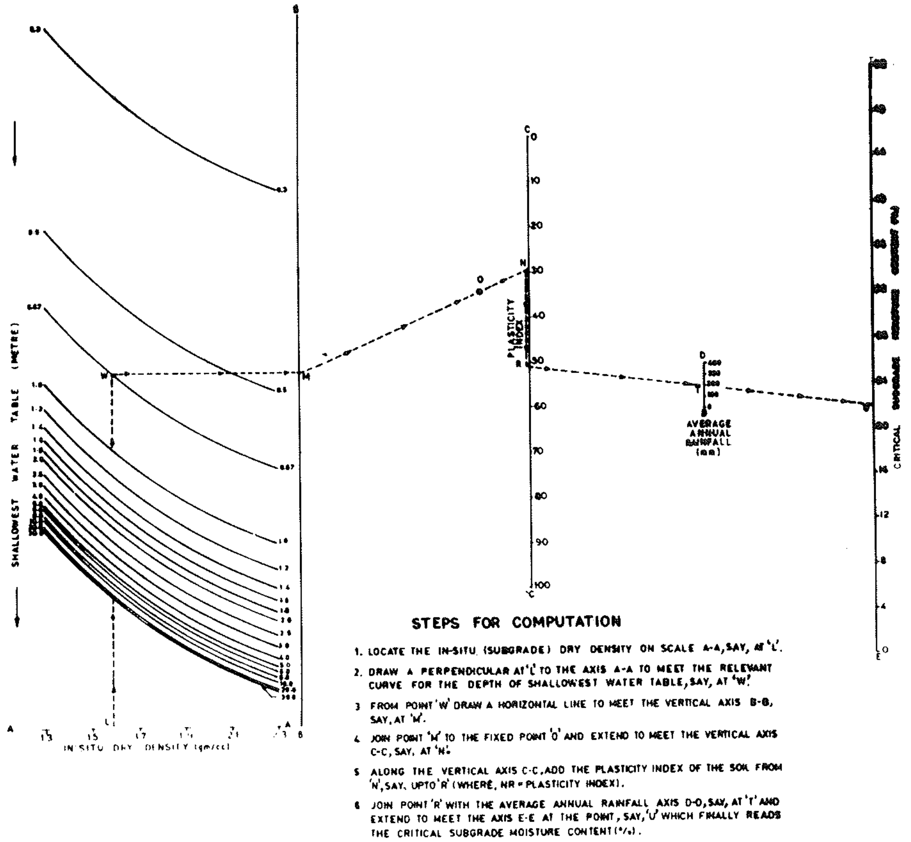


FIGURE 17 Nomograph for the computation of critical subgrade moisture content.

However, in regions of high rainfall and water-logged regions, 4-day soaked CBR values should be taken for design purposes. The nomograph shown in Figure 18 can be used to quickly estimate the soaked subgrade CBR values at Proctor density, based on particle size analysis. The following statistical expression relates the soaked CBR values of moorums (soil-gravel) with simple index properties (*7*).

Soaked CBR = $44.0A - 0.2B - 0.59C + 0.22D - 44.71$

where

A = dry density at standard Proctor compaction (gm/cm^3),
B = percent by weight of fraction retained on IS 2.36-mm sieve,
C = percent by weight of fraction passing IS 75 micron sieve, and
D = plasticity index.

A comprehensive survey of over 200 existing rural roads (after about 5 years of service) was undertaken in regard to the traffic volume and composition, pavement thickness and composition, subgrade and climatic conditions, and serviceability level. The data collected from this survey were analyzed. Then, based on the parameters of the traffic index and the subgrade strength index (correlated with CBR), two sets of pavement design curves (Figure 19) were developed for two categories of rural roads. The design curves depended on the following minimum levels of acceptable serviceability. The depth of rutting and transverse slope variance are 5 to 15 mm and 5 to 20, respectively, for a Category 1 road, and 10 to 40 mm and 20 to 25, respectively, for a Category 2 road.

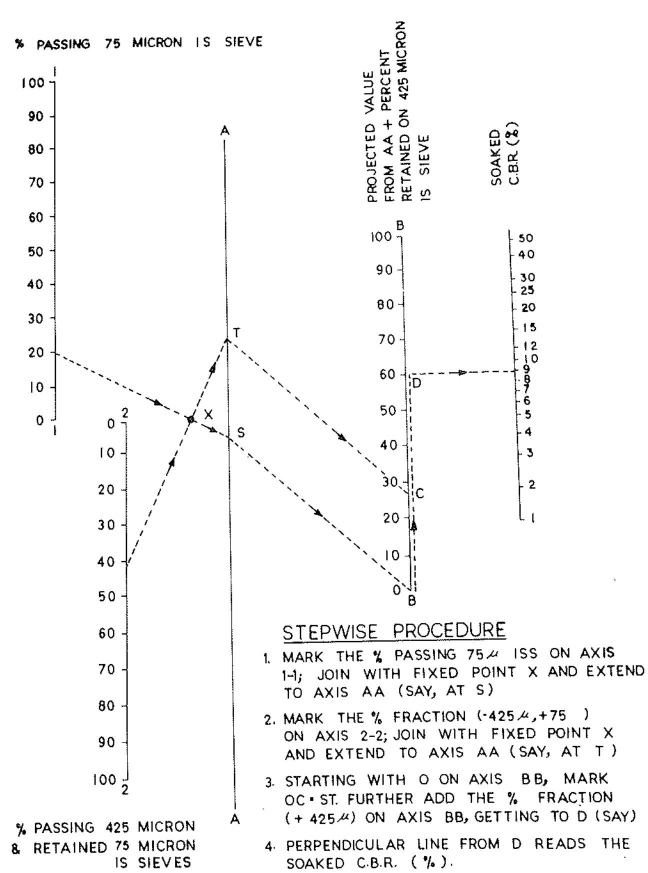
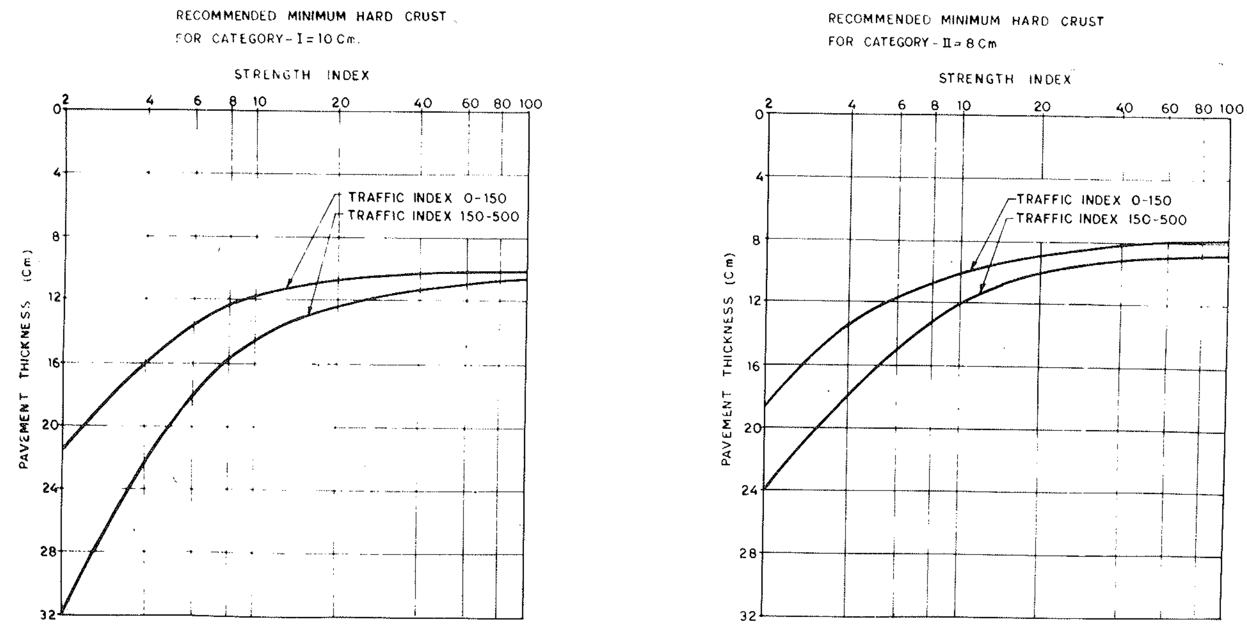


FIGURE 18 Nomograph for the computation of soaked CBR value from sieve analysis data.



DESIGN CURVES FOR RURAL ROADS CATEGORY-I **DESIGN CURVES FOR RURAL ROADS CATEGORY-II**

FIGURE 19 Pavement design curves for rural roads.

Category 1 roads are associated with relatively high speeds and traffic volume. This is to be expected on roads that connect large villages to market and growth centers, and roads that carry a relatively high percentage of pneumatic-tired commercial vehicles. Category 2 roads are associated with relatively low traffic volumes and slow-moving traffic, generally with a high percentage of animal-drawn carts. These roads connect small villages and hamlets.

SPECIAL PAVEMENT SYSTEM FOR SANDY REGIONS

Conventional road construction materials are not locally available in regions of sandy terrain such as in western Rajasthan in India, and have to be brought over long leads. The local soil is very fine and poorly graded sand. Dhandla, which is a softer variety of a calcareous aggregate, is available in pockets here and there. Water is generally very scarce. In view of the scant population and other related factors, the traffic generally is not heavy. Work was undertaken at the CRRI to determine if a special pavement system could be developed to suit such conditions (*8*). This work led to the development of a new pavement system that is composed of interconnected precast blocks.

This pavement system involves the use of two sets of blocks. Hexagonal blocks are used for the interior of the road and pentagonal blocks are used at the edges to obtain lanes of a uniform width. The blocks are ribbed and provided with a special dowel-sleeve arrangement for interconnection and load transfer. The blocks are precast at a local site, cured, and either stored or used immediately.

The blocks are placed directly on the prepared formation and subgrade of local dune sand. No other pavement component exists. The blocks are tightly seated next to each other with as few gaps as possible on a properly densified formation such as is used in conventional road construction. Although sand can be filled from the top through the holes that were provided for this purpose, it is best to use light vibratory compaction for seating. Each interior and edge block weighs about 210 kg and can be readily placed by three persons. A light, truck-mounted crane can also be used to place the blocks.

After the blocks are placed and properly seated, the sleeves are moved forward with a small hand tool to interconnect dowels in adjacent blocks. The small hollows over the sleeves can be left as they are or can be filled with lean concrete. Individual blocks can be replaced whenever necessary. The whole pavement section can also be dismantled and used elsewhere.

The pavement system was tested in the laboratory under both static and repetitive loading, using a specially prepared sand bed (Figure 20). The results confirmed the high structural efficiency that was built into the system for the conditions of its use. A design traffic intensity of 200 heavy vehicles a day has

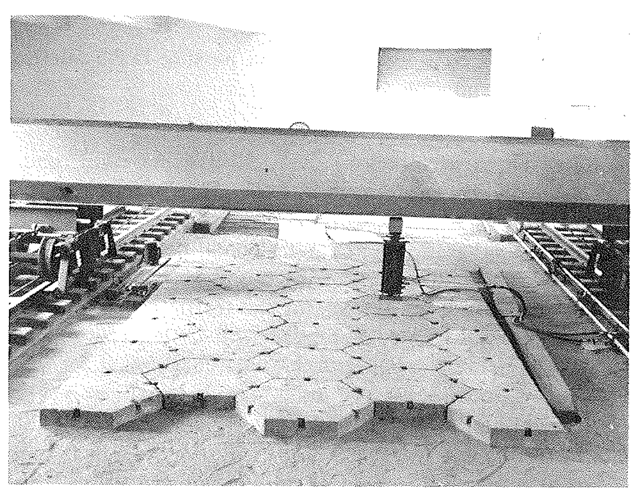


FIGURE 20 Assembly of precast blocks.

been assigned to the system for the time being. The system is also conceptually valid for other load intensities, and adjusted sections can be prepared for different loadings.

In terms of initial cost only, the new pavement system is slightly more expensive than a comparable flexible pavement with only a thin bituminous surfacing and is cheaper than conventional concrete pavements. Two road demonstration sections were recently constructed with this pavement system (Figure 21).

INTERMEDIATE TECHNOLOGIES

The road work in India continues to be performed with a relatively large degree of manual input (Figure 22). This is especially the case in the construction of low-volume roads, which is sometimes handled by agencies that are not well-equipped with expertise and infrastructure. It is recognized that road work continues to generate considerable employment, although it involves the use of some processes that are not so amenable to manual methods. There is also the question of added quality variations. Until large-scale mechanization is possible, technologies that constitute a good compromise between manual inputs and the quality and rate of production must be used.

Few, scattered jobs are created in the construction of low-volume roads. One important trend is the employment of agricultural implements that are readily available. Disc harrows (Figure 23) and moldboard plows that are towed behind a tractor are used for digging and pulverization. The mechanical power unit available in the form of tractors in rural regions provides for the use of several other low-cost items, such as the Rotavator or Rotiller (Figure 24) as mixers, towed rollers, and water bowsers.

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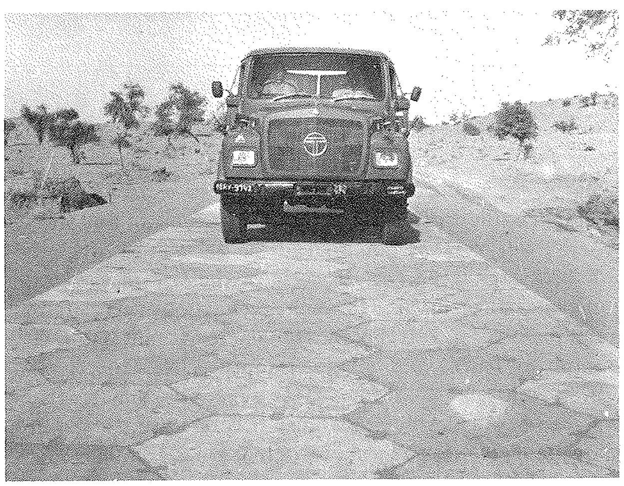

FIGURE 21 A demonstration stretch in which a special pavement system was adopted for sandy areas.

FIGURE 23 Disc harrows for pulverization of soil clods.

FIGURE 22 Manual methods in soil stabilization work.

FIGURE 24 Tractor-towed Rotiller for mixing soil with stabilizer.

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Physical and Operational Characteristics of Rail-Highway Grade Crossings on Low-Volume Roads

RONALD W. ECK AND RAJENDRAN SHANMUGAM

The National Rail-Highway Crossing Inventory and Federal Railroad Administration accident files were analyzed to compare low-volume road grade crossing characteristics with those of their higher-volume counterparts. Other objectives included the determination of accident rates, accident proportions, and effectiveness factors for low-volume road grade crossings and the comparison of these with other grade crossings. Results generally confirmed the hypothesis that low-volume road grade crossing characteristics are significantly different from those of higher-volume road grade crossings. The differences were more evident for physical characteristics than for operational characteristics. Accident rates, in which exposure was incorporated, at low-volume road grade crossings were much higher than those at higher-volume road grade crossings. There were also significant differences in accident proportions between low-volume road and higher-volume road grade crossings. Effectiveness factors for low-volume road grade crossing upgrades were different from those used in the U.S. Department of Transportation Resource Allocation Model. Flashing lights to gates upgrades were more effective for higher-volume road grade crossings (70 percent) than for low-volume road grade crossings (51 percent). However, upgrades from no signs or crossbucks to stop signs were more effective at low-volume road grade crossings (73 percent) than at higher-volume road grade crossings (59 percent).

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Potential conflicts can arise in the intersections of any traffic streams. However, the potential for conflicts at rail-highway grade crossings is unique. Because of the size of the train, significant changes in speed through deceleration or acceleration are not possible. Its travel path is limited to the rails. However, automobiles, trucks, and buses can stop, accelerate, decelerate, or turn in reasonable distances. Trains therefore must be given the right-of-way at grade crossings. It is the traffic engineer's responsibility to inform the motorist that a grade crossing exists and to alert drivers to the presence of trains so that drivers can take appropriate action.

Grade crossing warning devices include signs and signals on or adjacent to the highway approach to a rail-highway grade crossing. These traffic control devices can be classified as either active or passive devices (*1*). Passive devices include signs, pavement markings, and crossing illumination that identify and direct attention to the location of a grade crossing. Active devices include flashing lights and gates that are activated by the train to inform motorists of the approach or presence of trains on grade crossings. Gates have proven to be the most effective warning device in use because they provide a visible, if not physical, barrier between motor vehicles and the tracks.

The traffic engineer's job is to select the appropriate warning device for a given situation. Obviously, the ideal solution would be to install gates at all rail-highway grade crossings. However, because budget limitations make this impractical, the use of gates is usually reserved for the most dangerous crossings, and less effective devices are installed at other locations.

Between 1974 and 1985, approximately $900 million in federal aid safety funds were spent to provide active warning devices at nearly 22,000 crossings (*2*). Today, many of the most hazardous crossings have been improved and there is concern that a point of diminishing returns is being reached. Under this program, low-volume crossings are rarely reviewed by diagnostic teams. Any work performed at these crossings is usually limited to the installation of crossbucks and advance warning signs. However, recent statistics reveal that approximately half of the annual fatalities occur at low-volume crossings at which active warning devices may never be practicable.

Interest in low-volume crossings has recently grown. Federal Highway Administration Demonstration Project 70, "Railroad Crossing Corridor Improvements," was developed to encourage state highway agencies to expand their current programs to encompass many more crossings each year (*2*). Low-cost improvements are also emphasized at the types of crossings that are not currently being addressed.

The problem of selecting an appropriate warning device applies to all grade crossings, from those located on high-volume urban roads to those on low-volume rural roads. A low-volume road (LVR) is defined as a road with an average daily traffic volume of less than 400 vehicles per day (vpd). Although over 60 percent of the rail-highway grade crossings in the United States are on low-volume roads, the greatest number of grade crossing accidents occur at crossings on higher-volume roads. It is at these locations that exposure (the product of train volume and traffic volume) is the greatest. These crossings therefore have received the most study and funding.

Although traffic volumes are substantially lower, the potential for accidents at low-volume road grade crossings can still be great. Because of the low design standards that are typically used and a lack of maintenance, the crossing surface may be poor, which could contribute to the danger of vehicles stalling on the crossing. The roadway is frequently designed to minimum standards, which can create awkward horizontal and vertical alignments that contribute to sight distance problems. Because of funding constraints, pavement markings and signing at LVR grade crossings may not always meet recommended guidelines.

It has been said (*3*) that safety problems develop because many drivers do not frequently encounter trains at a particular crossing, and therefore expect the absence rather than the presence of trains. Low-volume roads can be especially vulnerable to this problem because the exposure between automobiles and trains is very low. Drivers on low-volume roads rarely expect to see other motor vehicles, let alone a train at a grade crossing.

A great volume of research has been conducted in recent years in regard to this problem (*4-12*). Studies performed have included the development of more effective crossing traffic control devices, formulation of accident prediction equations, and development of models for the optimal allocation of limited grade crossing resources. Virtually all of these studies have dealt with grade crossings in general and have not distinguished between low-volume and higher-volume facilities. However, as was stated earlier, the condition of and associated accidents at LVR grade crossings are not necessarily the same as those of higher-volume road (HVR) grade crossings. Therefore, agencies responsible for local roads may not in fact be maximizing safety and minimizing cost if they base their decisions solely on the information that currently exists on crossings in general. Information must be obtained on the physical and operational characteristics of LVR grade crossings and their associated accident experience. These data should be analyzed to determine if current procedures to predict grade crossing accidents and quantities measured for resource allocation are applicable to low-volume roads. If not, perhaps low-volume roads should be considered a separate category of grade crossing.

STUDY OBJECTIVES

A study was undertaken to analyze the National Rail-Highway Crossing Inventory data base and the Federal Railroad Administration (FRA) accident data files. The overall objective of this analysis was to compare the physical and operational characteristics of railroad-highway grade crossings on low-volume roads (ADT $\leq$ 400 vpd) with those of other classes of highways to provide assistance to road agencies involved in LVR grade crossing decision-making. Specific objectives of the research were as follows:

- To compare the physical characteristics of LVR crossings with those of other crossings, namely
  - Angle of crossing,
  - Number of tracks,
  - Highway pavement type,
  - Pavement markings,
  - Advance warning signs, and
  - Crossing surface type;
- To compare the operational characteristics of LVR crossings with those of other crossings, namely
  - Train movement,
  - Train speed,
  - Number of trains, and
  - Proportion of trucks;
- To compare the accident experience at LVR crossings with that of other crossings, namely
  - Vehicle position,
  - Position of train,
  - Circumstances,
  - Hazardous material involvement,
  - Severity,
  - Motorist action, and
  - Visual obstructions; and
- To analyze these results to determine whether currently used procedures and quantities for grade crossing accident prediction and resource allocation are appropriate, or whether crossings on low-volume roads should be considered a separate category. Effectiveness factors were examined as a specific parameter in this regard.

DATA ANALYSIS

The National Rail-Highway Crossing Inventory data base and the FRA accident data files from January 1, 1975, to December 31, 1981, were used in this study. The appropriate magnetic data tapes were obtained from the FRA.

Inventory Data

The original inventory file contained data for 213,907 public, at-grade rail-highway crossings. This file was divided into LVR and HVR crossings based on the highway traffic volume. As was stated earlier, LVR crossings were defined as those with an ADT of less than 400 vpd. New data files were named

LOWVOL and HIGHVOL, respectively. The original inventory data file contained 76 variables that ranged from the most important variables for the purposes of this study, such as crossing identification number, crossing angle, traffic volume (ADT), number of trains per day, and warning device type, to less important variables, such as number of bells and availability of commercial power. The new data sets contained only the 23 variables that were deemed necessary for this study.

Preliminary analysis revealed the data set LOWVOL contained 124,035 public grade crossings, and the data set HIGHVOL contained 89,672 public grade crossings. The LOWVOL data set was further divided into four different classes, A, B, C, and D, that correlated with ADT levels of 0 to 100, 101 to 200, 201 to 300, and 301 to 400, respectively. This subdivision was made to determine if any differences existed between classes of LVR grade crossings. The number of crossings in each volume class is shown in Table 1.

Accident Data

The FRA accident data file contained 93,226 accidents during the period January 1, 1975, to December 31, 1981. The accident data base contained 75 variables ranging from more important variables, such as total killed, total injured, type of accident, and visibility conditions, to less important variables, such as the county and state in which the accident occurred.

In order to create separate accident data sets for LVR and HVR crossings, the accident file had to be merged with the respective inventory data file (LOWVOL and HIGHVOL). New accident data sets were created for LVR and HVR crossings and named LVRACC and HVRACC, respectively. They were also stored on magnetic tape. The LVRACC data set contained 20,790 accidents and the HVRACC data set contained 51,257 accidents. The total number of accidents does not equal 93,226 because the original accident data file contained 7,932 accidents for 1982 (not included in this study) and because the remaining 13,247 accidents occurred either at private crossings or crossings that were not at-grade.

Accident characteristics were established by analyzing the new, merged data sets, LVRACC and HVRACC. The accident characteristics of LVR and HVR crossings were then compared. Accident rates, which included vehicle and train exposures, were also computed for different crossing characteristics, such as angle of crossing, vehicle speed, surface type, and other variables of this nature.

PHYSICAL AND OPERATIONAL CHARACTERISTICS OF LOW-VOLUME ROAD GRADE CROSSINGS

Physical Characteristics

A review of the literature indicated that physical characteristics were the principal contributing factors to grade crossing safety problems. Physical characteristics include angle of crossing, number of tracks, road surface, presence of advance warning signs and markings, and crossing surface type.

Angle of Crossing

The inventory file groups the angle of crossing into three categories: 0 to 29°, 30 to 59°, and 60 to 90°. An analysis of the frequency of the three categories of angle of crossing for the four classes of LVR grade crossings showed little difference between classes in terms of proportion of crossings in each angle category. Approximately 80 percent of the crossings were in the 60 to 90° category, 15 percent were in the 30 to 59° category, and 5 percent were in the 0 to 29° category. Similar results were obtained when the crossing angle characteristics of LVR grade crossings were compared with HVR grade crossings and with all crossings. The large proportion of grade crossings that had an angle of intersection between 60 and 90° was expected because 90° is the preferred angle of crossing in terms of minimizing human error and maximizing sight distance.

Number of Tracks

The inventory file gives the number of tracks for each grade crossing. Although as many as eight tracks per grade crossing exist, crossings with more than four tracks per crossing account for less than 0.5 percent of the total number of grade crossings. For this reason, the analysis considered only the data for grade crossings with up to four tracks. Note that the 0-track category represents crossings that do not have any main tracks. In other words, the tracks that do exist are used only for switching and the passing movement of trains.

Some differences existed between the volume classes in terms of proportion of number of tracks. The proportion of single-track crossings decreased with an increase in volume class, and the proportion of two-track crossings increased with an increase in volume class. As was expected, single-track crossings were

TABLE 1 NUMBER OF PUBLIC GRADE CROSSINGS IN DIFFERENT ROAD CLASS CATEGORIES IN THE NATIONAL RAIL-HIGHWAY CROSSING INVENTORY FILE

Low-Volume Road Crossing Category	Average Daily Traffic Volume	Number of Grade Crossings
A	0-100	76,279
B	101-200	19,939
C	201-300	19,330
D	301-400	8,487
Subtotal	--	124,035
Higher-Volume Crossings	>400	89,672
Missing Data	--	200
Total	--	213,907

predominant in the LVR grade crossing category. A difference also existed between vehicle volume categories and the number of tracks per crossing. Eighty-four percent of the LVR grade crossings had one track compared to 68 percent of the HVR grade crossings; this difference was significant. Note that the 0-track category occurred twice as frequently for HVR grade crossings as LVR grade crossings. This is because only switches are associated with many urban crossings that fall under the HVR category.

Road Surface

A notable finding of this analysis was that only 47 percent of the LVR grade crossing surfaces were paved, compared to 96 percent of the HVR grade crossings. An analysis of the LVR crossing data indicated that as the ADT increased, the proportion of crossings with paved surfaces also increased, which was expected. For Class A crossings, 29 percent of the road surfaces were paved, compared to 86 percent for Class D crossings. Such differences, though not necessarily of this magnitude, were expected and are consistent with the hypothesis that more HVR grade crossing surfaces are paved than LVR grade crossing surfaces, as shown in Figure 1. Several reasons exist for this situation. Higher-volume roads need to withstand more wheel passes and generally higher loads than low-volume roads. Low-volume roads also are usually of secondary importance when funds are allocated for construction or improvement. Whether or not a crossing is paved is taken into account in the DOT formula; it was found to be a significant factor only for crossings with passive devices.

Crossing Surface Type

Like road surfaces in general, a rough crossing surface can cause changes in driver behavior, such as a reduction in speed to negotiate the crossing. Although nine different types of grade crossing surfaces were defined in the crossing inventory, section timber, full wood plank, asphalt, and unconsolidated type of surfaces comprise 98 percent of all grade crossing surfaces. Therefore, data summarized in this discussion and the accompanying figures relate only to the four previously mentioned surface types.

The frequency of surface type by volume class for LVR crossings was then determined. Asphaltic surfaces predominated in the LVR grade crossing category. However, for Class A roads, the unconsolidated type (32 percent) was about as common as the asphalt surface (31 percent). This is because most Class A roads are unpaved and carry lower loads and volumes than roads of a higher class.

The frequency of crossing surface type by highway volume condition is shown in Figure 2. In the higher-volume category, asphalt surfaces are once again the dominant surface type. The difference is significant not only between the low-volume and higher-volume road categories, but also between the different classes in general. Such differences were expected.

Advance Warning Signs

Advance warning signs are intended to inform the motorist in advance of the existence of the grade crossing. The absence of advance warning signs would probably increase the likelihood of an accident at the grade crossing. The *Manual on Uniform Traffic Control Devices* (MUTCD) states that an advance warning sign should be used on each roadway in advance of every grade crossing, except on low-volume, low-speed roadways that cross minor spurs or other tracks that are infrequently used (*13*).

An analysis of the inventory data indicated, as expected, that a large proportion of higher-volume road grade crossings (55 percent) are equipped with advance warning signs compared to only 37 percent for LVR grade crossings. The difference of 18 percent is significant.

At least some of the difference in the percentages of crossings with advance warning signs could be attributed to the fact that different jurisdictions have maintenance responsibility for the highways. A smaller jurisdiction, which is more likely to maintain a low-volume road, is less likely to do as much signing as a state or a large municipality that normally maintains a higher-volume road or street.

The use of highway pavement markings is another way to provide drivers with advance warning at grade crossing approaches. Two types of markings exist at grade crossing approaches: a stop line and a railroad (RR) symbol. The inventory file lists the markings under the four categories of

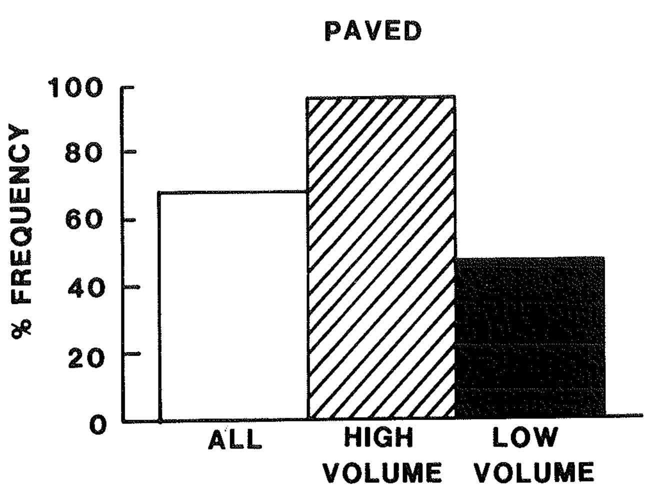

FIGURE 1 Frequency of paved road surface by highway volume condition at grade crossings.

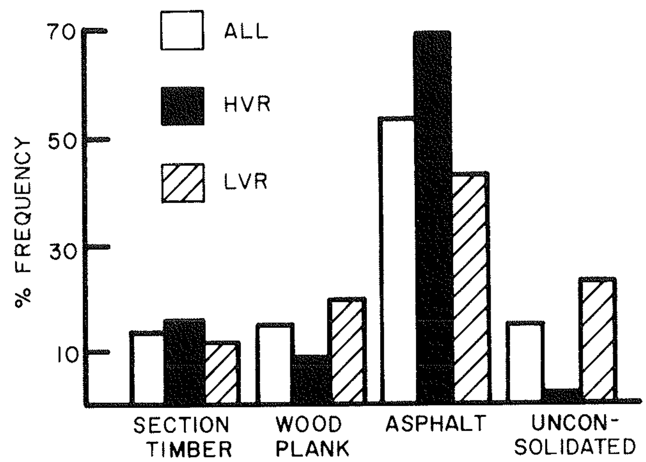

FIGURE 2 Frequency of crossing surface type by highway volume condition at grade crossings.

stop line, railroad symbol, no markings, and stop line and railroad symbol.

As shown in Figure 3, 91 percent of the LVR crossings do not have markings, compared to only 68 percent for the HVR grade crossings; the difference is significant. A number of possible explanations exist for this finding. Most grade crossings in the LVR category tend to be unpaved and therefore do not have any markings. Approach speeds are also less likely to meet the MUTCD standard of 40 mph or greater, and active devices are much less likely to be present at a low-volume crossing. Finally, the jurisdiction with maintenance responsibility may once again play some part.

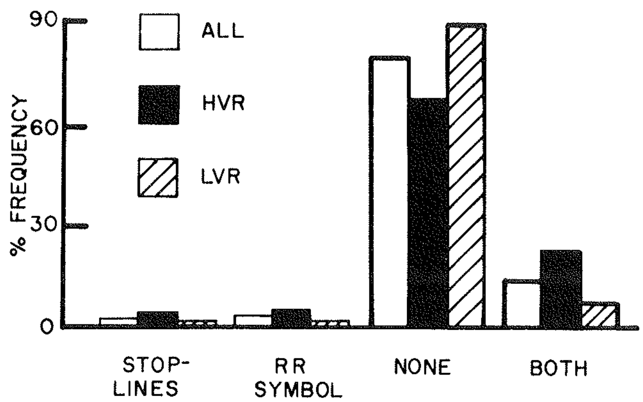

FIGURE 3 Frequency of highway marking by highway volume condition at grade crossings.

Operational Characteristics

A review of the literature indicated that operational characteristics such as train volume, train speed, warning type, and truck volumes are some of the most significant characteristics in regard to accidents at rail-highway grade crossings. The study examined the important operational characteristics of LVR crossings and compared them with those of HVR crossings.

Train Volume

A motor vehicle and a train obviously must be present at a grade crossing for an accident to occur. The higher the volumes of either or both, the greater the chance for a conflict. This fact has been well recognized by researchers; almost every accident prediction formula and hazard index formula uses train and vehicle volumes as basic inputs (*1*).

The inventory file groups train volume data in four separate categories: day through trains, day switch trains, night through trains, and night switch trains. Through trains do not start or terminate at or near the vicinity of the grade crossing. Switch trains start or terminate at or near the vicinity of the grade crossing.

The analysis indicated that zero to nine trains travel over 90 percent of the crossings a day. A chi-square goodness-of-fit test indicated no significant differences between the data for LVR crossings and those for HVR crossings. Statistical tests for various aggregations of train movement data yielded similar results. This implies that accident prediction and hazard index formulas based only on train movement for all grade crossings can be applied equally well to LVR grade crossings.

Train Speed

Train speed is one of the most important operational characteristics that determine the severity of a grade crossing accident. In order to simplify access to the inventory data, train speeds were grouped into units of 10 (i.e., 0 to 9 mph, 10 to 19 mph, etc.). For HVR grade crossings, 30.8 percent had a maximum train speed of 10 to 19 mph; this was the largest single speed group. For LVR grade crossings, 30 to 39 mph was the largest single maximum speed group, with a frequency of 22.8 percent. In the minimum train speed group for both low-volume and higher-volume road grade crossings, 0 to 9 mph (almost standing) was the single largest speed group (42.8 and 60.6 percent, respectively).

A chi-square goodness-of-fit test was performed to compare the train speed distributions between LVR, HVR, and all grade crossings. The results indicated no significant differences between the train speed distributions.

Proportion of Trucks

Truck volumes are considered to be one of the important operational characteristics that influence grade crossing safety. Crossings with a high proportion of trucks should be given careful consideration because of mandatory stopping laws (for trucks carrying hazardous materials) at grade crossings and because of the contributing role truck characteristics play in grade crossing accidents.

The inventory file contains truck data as a proportion of the traffic volume. For all three highway volume categories, crossings with truck proportions over 20 percent were negligible compared to crossings with truck proportions of less than 20 percent. A chi-square goodness-of-fit test indicated no significant differences in truck proportions for the three highway volume categories.

Warning Device Type

Warning devices are used at rail-highway grade crossings to identify and direct attention to the location of the crossing. Some devices detect the presence of a train at or near the crossing, which allows motorists and pedestrians to take appropriate action. The inventory file contains the warning device type data under eight classes. These classes and the proportion of grade crossings under each class, by highway volume category, are shown in Figure 4.

As expected, there was a significant difference of over 5 percent in warning device type distribution between LVR and HVR grade crossings. Eighty percent of the LVR grade crossings are protected by crossbucks only, whereas only 39 percent of the HVR crossings are protected by crossbucks only. Only 6 and 3 percent of the LVR grade crossings are protected by flashing lights and gates, respectively, whereas the corresponding values for HVR crossings are 31 and 15 percent, respectively. Overall, 48 percent of the HVR crossings are

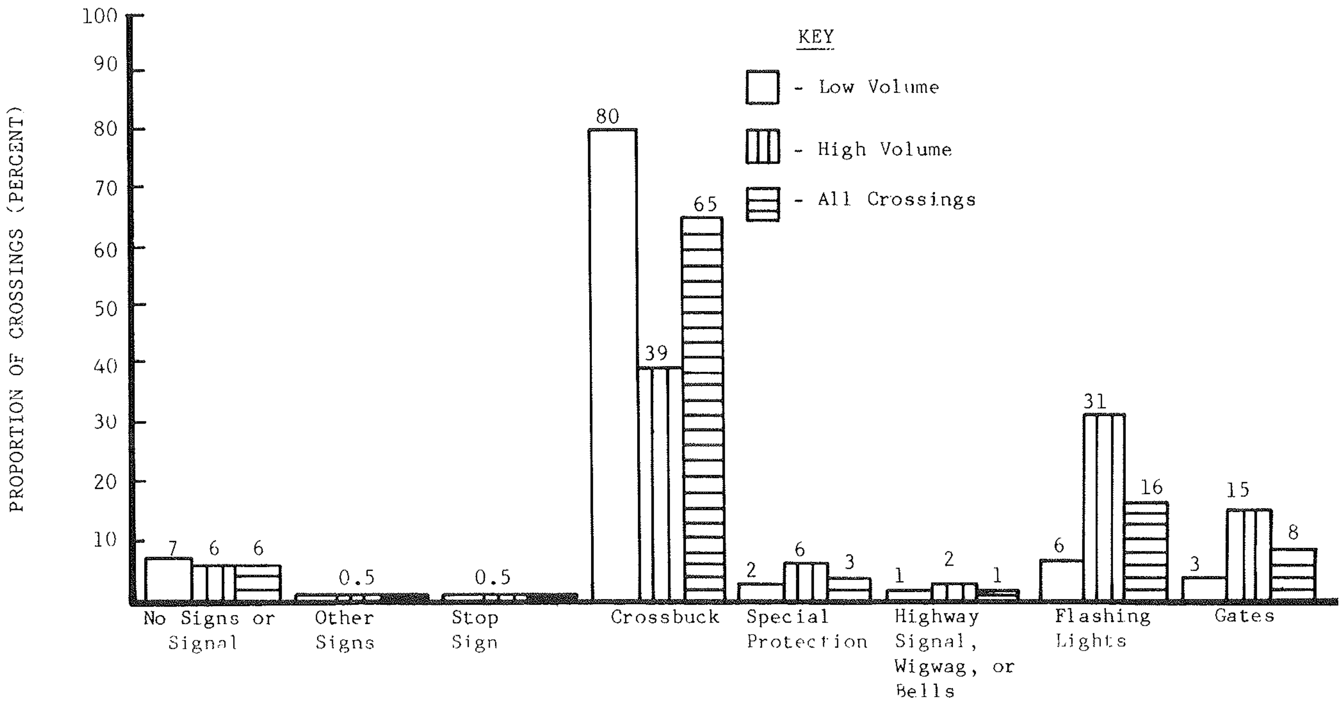

FIGURE 4 Proportion of crossings, by warning device type, for various highway volume conditions.

protected by active devices compared to 10 percent of the LVR crossings. Such great differences are significant and are a result of the importance placed on HVR grade crossings over LVR grade crossings in reducing accidents. These results indicate that when working with aspects of crossings in which the warning device type is significant, such as in the use of the hazard index or accident prediction formulas, attention should be given to stratifying LVR and HVR grade crossings. Results that are based on all grade crossings in general may not be valid because of the significant differences between the type of grade crossing protection used at LVR and HVR grade crossings.

ACCIDENT CHARACTERISTICS AND EFFECTIVENESS FACTORS

Accident Rates

Most literature in the area of rail-highway grade crossings presents accident data in terms of accidents/crossing/yr. Although this could be loosely interpreted as an accident rate, strictly speaking it is not, because the ratio does not incorporate a measure of exposure. Exposure refers to data about the population at risk, such as train and traffic volume. Exposure data are important because they are critical to the calculation of the actual likelihood of an accident. The accident rates presented in this section therefore include vehicle and train exposure; the units are expressed as accidents per vehicle-train per day (acc/v-t/d) times 10^{-8}.

Accident rates can be computed for any physical or operational characteristic at grade crossings. Based on a preliminary analysis, accident rates appeared to represent accident patterns very well for physical characteristics, but not so well for operational characteristics. Physical characteristics, such as angle of crossing, vary between grade crossings, which makes it possible to compare accident rates between grade crossings. However, operational characteristics, such as speed of train at time of accident, vary within a grade crossing. Comparisons therefore are made within the grade crossing instead of between grade crossings, as was desired. Therefore, accident rates for LVR grade crossings were computed for some of the most important physical characteristics, such as angle of crossing, crossing surface type, and presence of advance warning signs and pavement markings. These accident rates were then compared with those of HVR grade crossings.

In general, the accident rates at HVR grade crossings were considerably lower than those of LVR grade crossings. This was expected because of the superiority that HVR crossings have in terms of geometry, physical conditions, and warning device type.

Accident rates for LVR and HVR grade crossings for the three crossing angle groups are shown in Figure 5. Accident rates for the HVR grade crossings were about the same (2.2×10^{-8} acc/v-t/d) for each of the three groups of angles. Accident rates were somewhat different between the three angle groups for LVR grade crossings. Angle group 60 to 90° had the highest accident rate of 20.8×10^{-8} acc/v-t/d.

Of the four dominant crossing surface types, section timber (24.7×10^{-8} acc/v-t/d) and unconsolidated crossing surfaces (22.7×10^{-8} acc/v-t/d) had the highest accident rate in the LVR category. Accident rates for rubber and concrete type crossing surfaces in the LVR grade crossing category were very high perhaps because of the fact that drivers can traverse them at high speeds. However, these conditions comprise less than 1 percent of all LVR grade crossings. For the HVR category, section timber (2.7×10^{-8} acc/v-t/d), full wood plank (2.7×10^{-8} acc/v-t/d), and asphalt (2.5×10^{-8} acc/v-t/d) crossing surfaces had the highest accident rates.

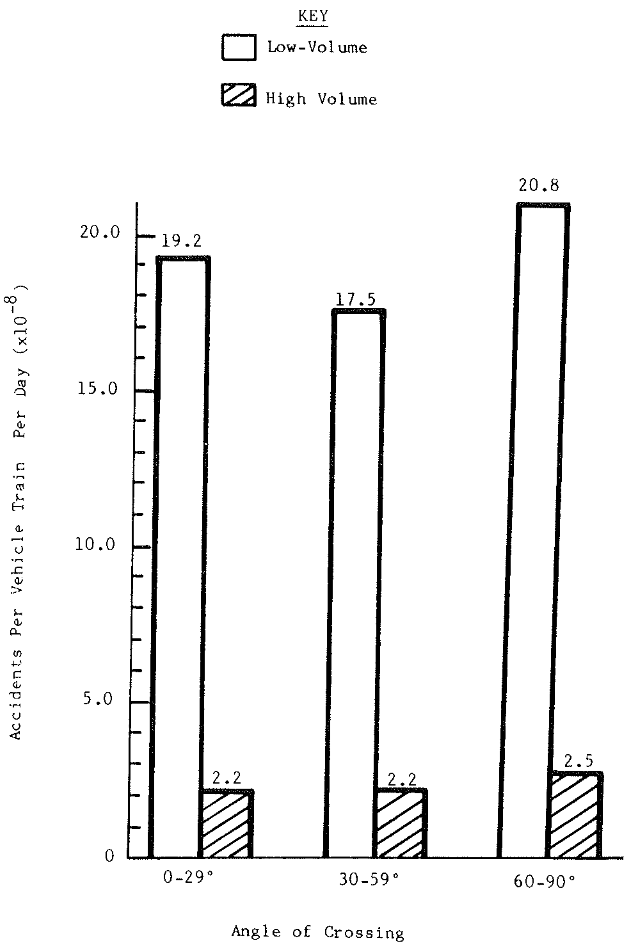

FIGURE 5 Accident rates for low-volume and higher-volume road grade crossings by crossing angle.

Accident rates were higher at LVR crossings at which no advance warning signs were present than when advance warning signs were present (22.5 vs. 16.4×10^{-8} acc/v-t/d). It is likely that grade crossings on many low-volume roads may not be clearly visible to motorists for reasons that were outlined earlier. Motorists therefore place more reliance on advance warning signs on low-volume roads than on higher-volume roads. The absence of advance warning means that the crossing might not be detected as readily and more accidents might be expected.

Accident rates were about the same for HVR grade crossings, regardless of the presence or absence of advance warning signs (about 2.5×10^{-8} acc/v-t/d). Similar results were found for pavement markings. However, crossings with both types of pavement markings (stop line and RR symbol) had higher accident rates than those with only one type of pavement marking.

Accident Proportions

Accident proportions are another way of representing accident patterns for most operational characteristics and some physical characteristics at grade crossings. Low-volume road grade crossing accident proportions were computed for type of vehicle, position of vehicle, vehicle speed, train speed, type of accident, number of people killed or injured, visibility at the crossing, obstruction to view, and presence of hazardous materials. These proportions were compared with HVR grade crossing accident patterns. As part of this analysis, tests for the differences between two proportions were performed to determine significant differences between LVR and HVR and grade crossing accident proportions.

An analysis was made of grade crossing accidents by type of motor vehicle involved. Although there are eight different vehicle categories, two categories, automobile and truck, accounted for nearly 90 percent of all accidents. Seventy-one percent of HVR crossing accidents involved automobiles compared to 63 percent of the LVR crossing accidents. Eighteen percent of HVR crossing accidents involved single-unit trucks compared to 25 percent of LVR crossing accidents. Both differences are significant. Contrary to initial expectations, truck-trailer combinations had a low accident proportion for both LVR and HVR grade crossings. It was expected that truck-trailer combinations would have high accident proportions because of the greater time required for a long vehicle to negotiate the crossing at slow speeds.

The pedestrian accident proportion at HVR crossings was twice as great as that for LVR crossings. This may be because many HVR grade crossings are located in urban areas and tend to have more pedestrian movement than LVR crossings, which are primarily located in rural areas.

As was expected for both LVR and HVR grade crossings, moving vehicle accidents comprise the largest proportion of accidents at 70 and 75 percent, respectively. The moving vehicle accidents for HVR crossings are significantly higher than those for LVR crossings. As was expected, stalled and stopped accidents on LVR crossings (15 percent for both) were significantly higher than those for HVR crossings (9 and 13 percent, respectively). This is probably a result of the large proportion of unpaved road surfaces and poor geometric conditions at LVR grade crossings, which force the vehicle to suddenly reduce speed and change gears, thereby increasing the chances for the vehicle to stall.

A significant number of grade crossing accidents (about 47 percent overall) occurred at low vehicle speeds (0 to 9 mph). However, the proportion of LVR crossing accidents at low vehicle speeds was significantly higher than that of HVR crossing accidents. This is probably a result of the poorer physical condition of the road and grade crossing.

Accident proportions for LVR crossings were more or less equal for train speeds up to 50 mph. The greatest proportion of accidents for HVR crossings (37 percent) occurred at train speeds of 0 to 9 mph. Accident proportions were significantly lower for LVR crossings at train speeds less than 20 mph. For train speeds of over 20 mph, the accident proportions of LVR crossings were significantly greater than those of HVR crossings. Accidents at LVR grade crossings tended to be more severe than those at HVR crossings.

The proportion of accidents that resulted from an obstruction of the driver's view was significantly higher at LVR crossings than at HVR crossings. This tended to confirm the hypothesis that sight distance restrictions are more prevalent at LVR crossings than at HVR crossings.

Although only about 5 percent of the accidents at both LVR and HVR grade crossings involved the presence of hazardous materials, the potential severity of such accidents must be recognized. The accident proportion at LVR grade crossings in which hazardous materials were present (6 percent) was significantly higher than that of HVR crossings (4.7 percent). Hazardous materials carriers might use low-volume roads either because of the presence of terminals or to deliver certain products such as agricultural chemicals, propane, or heating oils.

Effectiveness Factors

Effectiveness factors for different safety improvements are required in order to use the U.S. DOT rail-highway crossing resource allocation model. The effectiveness of a warning device is defined as the fraction by which accidents are reduced after it is installed. The resource allocation model considers three categories of warning device upgrades: passive to flashing lights, passive to flashing lights with gates, and flashing lights to flashing lights with gates. Previous effectiveness factor studies have considered all grade crossings in general (*5, 14-16*). However, because of differences in geometric design, road and crossing surface types, presence of advance warning, and other such variables between LVR and HVR crossings, the effectiveness factors for LVR crossings were expected to be different.

One of the limitations in examining the upgrade effectiveness of LVR crossings is the amount of data available. Because very few LVR crossings are equipped with active warning devices, the sample size is very small and the confidence intervals are very large. Two other primary upgrade types for LVR crossings are no signs to crossbucks, and no signs or crossbucks to stop signs. These may be more important types of upgrade, because most LVR crossing upgrades come under these two categories. The importance of these two primary upgrade types has been recognized in recent studies (*14, 15*).

Effectiveness values and confidence intervals for these five types of upgrades were calculated for LVR grade crossings, HVR grade crossings, and all grade crossings. The results are summarized in Table 2. Effectiveness factors for all grade crossings can be used as the base value because this is what is used by the resource allocation model for the three main categories of upgrades. Effectiveness factors for the LVR and HVR grade crossings can be compared with these base values to determine if any differences exist. Although some differences exist in the effectiveness factors of the first two categories, passive to flashing lights and passive to flashing lights with gates, they are within the confidence interval (CI) of all grade crossing effectiveness factors. The third category, flashing lights to flashing lights with gates, shows a variable effectiveness factor. An effectiveness factor of 51 percent for LVR crossings is very low compared to the 70 percent value for HVR crossings. The HVR grade crossing effectiveness factor falls within the CI of all grade crossing effectiveness factors of 69 percent (CI = 66 to 73 percent). However, the LVR effectiveness factor does not fall within the CI of all grade crossing effectiveness factors.

Such significant variation in the effectiveness factors can make a noticeable difference in how the resource allocation model is used. This illustrates the importance of analyzing LVR crossings separately from HVR crossings. Note that the CI for the LVR crossing effectiveness factor (30 to 72 percent) is very high, because the number of changes in this category is very low.

TABLE 2 SUMMARY OF RESULTS OF EFFECTIVENESS FACTORS BY HIGHWAY VOLUME CONDITION AND WARNING DEVICE UPGRADE CATEGORY

	Upgrade Category	Number of Crossings	Number of Before Accidents	Number of After Accidents	Number of Before Years	Number of After Years	Effectiveness Factor	95 Percent Confidence Interval
Low-Volume	P to FL	792	204	47	1896.6	1757.4	75.2	67.5-82.9
	P to G	967	432	52	2662.5	2091.6	84.7	80.4-89.0
	FL to G	200	70	27	556.7	435.5	50.7	29.7-71.7
	No Sign to C	2641	363	96	11575.9	3530.7	13.7	--
	No Sign/C to SS	110	17	2	382.4	164.0	72.6	--
	Total	4710	1086	224	17074.2	7979.1	--	--
Higher Volume	P to FL	1824	780	173	4854.1	3807.3	71.9	75.4-76.4
	P to G	1642	1128	137	4559.1	3643.1	84.8	79.7-89.9
	FL to G	1805	1307	321	5053.7	4122.9	70.1	66.7-73.5
	No Sign to C	1177	405	106	4884.3	1698.6	24.8	--
	No Sign/C to SS	44	19	3	124.7	47.1	59.1	--
	Total	6492	3639	740	19475.9	13319.0	--	--
All Grade Crossings	P to FL	2616	984	220	6750.6	5564.7	73.0	69.2-76.8
	P to G	2609	1560	189	7221.7	5738.7	84.9	82.7-87.1
	FL to G	2005	1377	348	5610.3	4558.3	69.2	65.8-72.6
	No Sign to C	3818	768	202	16460.1	5229.3	17.2	4.6-29.8
	No Sign/C to SS	154	36	5	507.0	211.1	66.8	36.2-97.4
	Total	11202	4725	964	36549.7	21298.1	--	--

P-Passive FL-Flashing Lights G-Gates C-Crossbucks SS-Stop Sign
--data not applicable

As was expected for all three highway volume conditions, stop signs were much more effective than crossbucks. Stop signs are also more effective at LVR crossings (73 percent) than HVR crossings (59 percent).

CONCLUSIONS AND RECOMMENDATIONS

The results of this study generally confirmed the hypothesis that low-volume road grade crossing characteristics are significantly different than those of higher-volume road grade crossings. The differences were more evident for physical characteristics than for operational characteristics. The only physical characteristic for which there was no significant difference between the two types of crossings was the angle of crossing. Operational characteristics such as number of trains, train speed, and truck volumes at grade crossings showed no significant differences between the two types of crossings.

The fact that there was no difference in the angle of crossing between LVR and HVR crossings should not be interpreted as meaning that the effect of angle of crossing in determining crossing safety should be the same for both volume classes. Many low-volume roads achieve a great crossing angle by introducing sharp horizontal curvature on the approaches. Although the crossing is recorded as a 90° angle, it does not function as one.

In general, accident rates at LVR grade crossings were much greater than those of HVR grade crossings. There were also significant differences between the two types of grade crossings for several different accident categories.

There were differences in effectiveness factors between LVR grade crossings and all grade crossings for several types of upgrades. These differences should be taken into consideration by low-volume road decision-making agencies. Because effectiveness factors are one of the major inputs in resource allocation, differences in effectiveness factors can make a difference in the outcome of a decision.

Based on the results of this study, two recommendations are made to assist road agencies involved in LVR grade crossing decision-making:

- Because of the great differences that were noticed in the physical and accident characteristics between LVR grade crossings and all other grade crossings, it appears appropriate to analyze LVR grade crossings separately.
- The application of accident prediction formulas and hazard formulas, which are derived by using all grade crossing characteristics, to LVR grade crossings should be approached with a considerable amount of engineering judgment.

During the course of this study, it was noticed that one of the most important grade crossing characteristics, sight distance, was not included in the inventory data. However, data for sight obstruction were available in the FRA accident data file. Additional useful data could be developed if information was available on the sight distances and directions of approach of vehicular and train traffic at grade crossings. Further study is warranted, perhaps involving field investigations, to determine the influence of these variables on accident experience. The development of a simple but meaningful way to incorporate such factors into the inventory data base appears to be appropriate.

It would also be desirable to record the width of the crossing surface. Each crossing surface should be at least as wide as the approach roadway and shoulders. One of the most common deficiencies observed during the corridor review process that was described earlier was that crossing surfaces were too narrow (*2*). The resulting exposed tracks could cause a low-speed vehicle to become stuck on the tracks or a high-speed vehicle to go out of control.

One limitation of this study was that the accuracy of the data base was not checked. Experience in several states indicates that the data base is in error in many cases, especially in regard to highway information. The principal reasons for the deficiencies include the number of data items in the inventory file, failure to update data in response to new signs and markings, and the poor quality of volume data on local roads. It is recommended that the accuracy of the data base be checked in any future studies of this type.

ACKNOWLEDGMENTS

Appreciation is expressed to John Halkias, who had implemented the grade crossing data base as part of an earlier project. Halkias also provided advice during the preliminary data analysis. The authors are grateful to Ray Lewis of the West Virginia Department of Highways for his helpful comments.

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Rock and Debris Slide Risk Maps Applied to Low-Volume Roads in Nepal

ALEXIS WAGNER, RAYMOND OLIVIER, AND EDUARDO LEITE

A discussion is provided of rock and debris slide risk mapping along low-volume road corridors in the foothills of Nepal. First, the results of a data compilation of the main factors leading to the failure of rocky and semi-rocky terrains are described. This research was conducted in Nepal on over 100 rock and debris slides. These data were developed into a rock and debris slide risk mapping method that was experimented with success along 300 km of road corridor sections in Nepal. The method is based on a superimposition of the geological, morphostructural, and slope maps of the road corridor in association with "weights" in percent to the most relevant factors leading to failure. The initial results are of a computerized risk mapping system that was applied to low-volume roads in mountainous developing countries. Because the initial data compilation revealed the structural factor to be a very crucial one, a test of a computerized structural risk map was operated on an already mapped road project section. This test was found to be consistent with the original risk map and revealed, with more accurate limits, similar locations of the risk areas in which slides actually occurred as predicted. Other advantages of the computer map are the systematic aspect of the process and the simulation flexibility of the unique parameters according to the observed field data and the local imposed conditions.

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A synthesis is presented of geological research concerning techniques for mapping the risks of rock and debris slides along low-volume road corridors in Nepal, and hydrological work in which digitalized elevations were combined with hydric data to yield hydrological balance maps (*1-6*). The goal is to create a computerized system for landslide risk mapping that is geared especially to low-volume roads and other alignments in mountainous developing countries. This work was commissioned by the Swiss National Fund for Scientific Research.

It is well known that careless construction of low-volume roads in mountainous developing countries causes heavy environmental damage and high maintenance costs. This presents constant challenges to the efficiency and liability of the projects themselves. For example, 5 percent of the total surface of landslides in Nepal is created by road construction (*7, 8*). The roads themselves cover a surface of about 15 km^2 in the Nepalese foothills, whereas the area sensitive to landslides is about 60,000 km^2. In Nepal, the construction of roads therefore creates conditions 200 times more likely to cause land movement than the average of other human activities and the natural tendency of the terrains to slide. The goal of the present work is to contribute to the alleviation of this worrying situation. It is hoped that by implementing a reliable method of accurately identifying alternate, safe alignments and pointing out sections in which specified techniques should be applied for construction and maintenance, significant progress will be made in this direction.

RESULTS OF THE FIELD-OBSERVED DATA COMPILATION IN THE FOOTHILLS OF NEPAL

The data compilation is concentrated on rock and debris slides of the translational type, which is the dominant form of land movement on slopes and within stream channels in the steep Nepalese foothills. The relatively sparse rotational type of slide is absent from this compilation.

A translational slide consists of a mass that progresses out or down and along a more or less planar, or wedge-shaped, surface. The slide material is either greatly deformed or consists of many semi-independent units. In Nepal, the material that constitutes these slides is usually topsoil mixed with fragments of bedrock that vary in size from gravel to large blocks. In fact, the slide material typically originates in the bedrock, but has been strongly altered by subtropical climatic conditions to the point that, in the upper layer, it constitutes a soil, properly speaking. The majority of these slides can be qualified as debris slides, or sometimes as rock slides.

Factors That Lead to Debris or Rock Slides: Collection of Data

According to field experience and knowledge of the subject, the following are the most relevant factors of the considered site:

- Relationship between the rock structure and the slope;
- Type of rock;
- State of weathering and crushing of the rock;
- Presence of faults, thrust faults, and folds;
- Hydrology and hydrogeology; and
- Morphology.

This collection of field data was conducted along sections of the Lamosangu-Jiri, Arniko, Muglin-Narajanghat, Raj Path, Prithvi-Rajmarga, and Siddharta road corridors; various rivers; and within various zones of Nepal (Figure 1).

The Structural Pattern of the Bedrock

The structural pattern of the bedrock is determined by the relation between the different geologic planes or beds within or adjacent to them. The stability of a rocky or sub-rocky slope or cut-slope depends greatly on the relation between the rock pattern and the plane of the slope itself.

An upper equatorial Schmidt projection of the bedrock structure was drawn for each slide. This projection allows the realization of a geotechnical model that enables one to easily determine whether freedom toward a free surface is possible, and, if necessary, to calculate the resulting failure and strength relation.

Freedom of movement on a geologic plane or on two intersecting geologic planes may occur when the plane or the intersection dips in a direction close to the slope of the site, but is less inclined than the slope itself. As stability assessments are made along further cut or eroded slopes, one should consider that freedom of movement also occurs when a geologic plane, or an intersection of two geologic planes, is parallel to the slope of the site. When a geologic plane is parallel to the slope of the site, the latter is called a structural slope. The intersecting geologic planes with freedom of movement result in a so-called wedge pattern (Figure 2). In subsequent angle references in the text, grads are used as angle units (100 grads equal 90°), because their use is more convenient in field practice and structure plotting.

When a wedge shows an intersection parallel or close to the direction of the slope (up to 35 grads on each side of the direction of the slope), it is defined as a central wedge. From 35 to 70 grads, it is defined as a lateral wedge. Beyond this value it is described as a very lateral wedge (Figure 3).

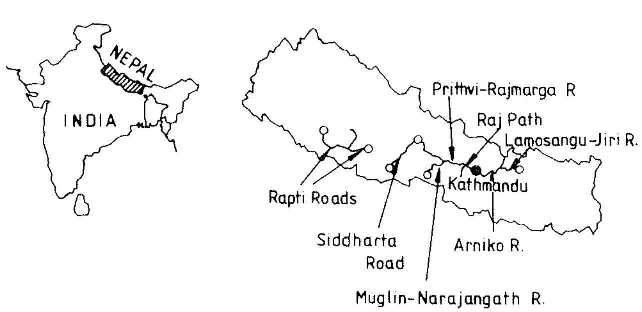


FIGURE 1 Location map.

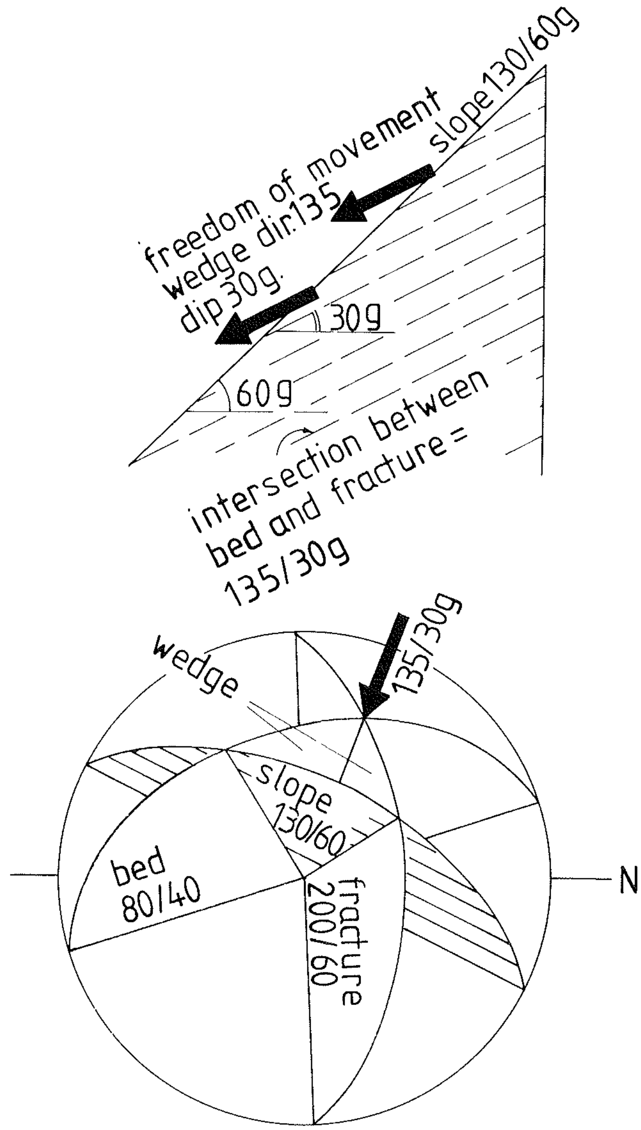


FIGURE 2 Central wedge and freedom of movement. The arrow represents the direction and dip of the structural wedge. The slope, indicated by S = 130/60, is nonstructural.

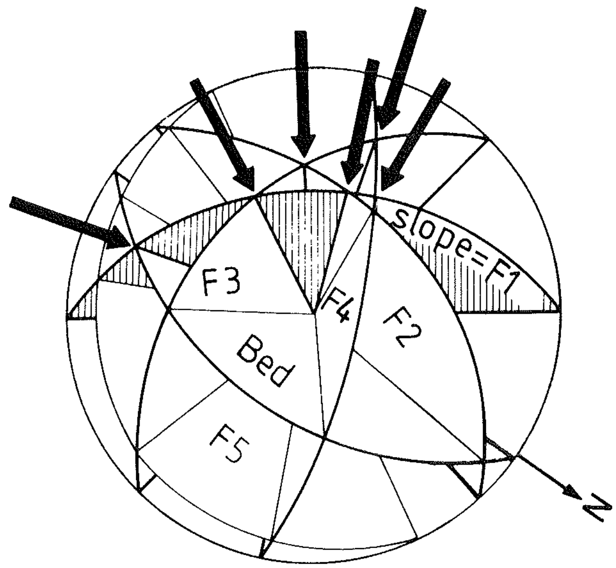


FIGURE 3 Central, lateral, and very lateral wedges. The arrows represent structural wedges. The slope indicated by S = F1 is controlled by fracture F1 and is therefore structural.

Other Factors

- The incline and direction of the slope are either a) the estimated angles before the slide occurred, or b) angles measured close to the slide within the undisturbed area.
- The density of geologic planes: high, medium, and low. This is an estimation of the number of layers, fractures, or other type of joint, per meter of a rock encountered when crossing a line perpendicular to each geologic plane.
- The type of rock and its state of weathering: highly weathered, fairly weathered, or sound.
- Presence of specific minerals subject to weathering (pyrite and chlorite) or increasing the risk of slide (sericite, micas, and graphite).
- The occurrence of seepages or springs within or adjacent to the slide. The proximity of seasonal or perennial gullies, rivulets, or rivers.
- The presence of faults or thrust faults within or near the slide areas.
- The morphology of the site: concave (coomb and depression) or convex (crest and ridge).
- The size of the slide and a description of its material and slide surface.

Results of the Field-Observed Data Compilation

The Structural Pattern

Particular care was given to the study of the structural pattern. As was stated earlier, experience has shown how important the relation between structure and a natural or cut slope is to stability.

The first working hypothesis was simple. A landslide occurring on a slope with only one central wedge (CW), for instance, a wedge approximately parallel to the direction of the slope and less inclined than the slope (or in a borderline case, equal to the slope), will create two new lateral slopes. But this slide will not create the preconditions necessary for slides to occur on the newly created slopes (Figure 4). A wedge pattern must first exist on the new slopes, after which lateral slides can take place. This cycle could repeat itself several times, provided that wedge patterns exist for each case (Figure 5). Whenever the structural projection of the rock in a site indicates the presence of a fan of wedges, the potential occurrence of a large slide is indicated.

The working hypothesis was confirmed by the compilation of field data in which the results showed that the surface area of a slide tends to increase with the number of wedges (see Figure 6). In fact, the following characteristics can be recapitulated:

- All of the rock slides resulting from the presence of a single wedge had surface areas smaller than 0.25 hectare.
- With 2 to 3 wedges, 70 percent of the slides remained inferior to 0.25 hectare.

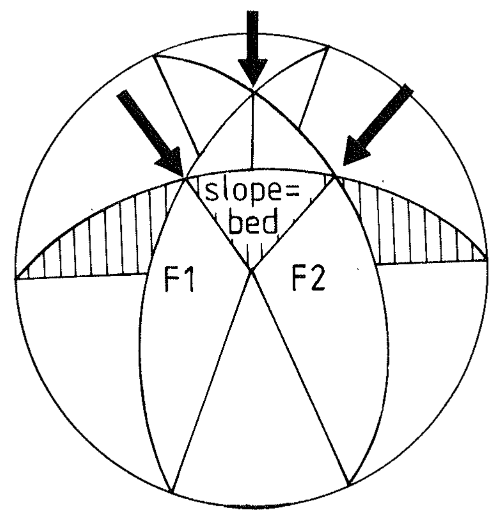


FIGURE 4 A central wedge acts as a keystone and frees the movement of lateral wedges. The arrows represent structural wedges. The slope S = Bed is controlled by the bed of the rock and is therefore structural.

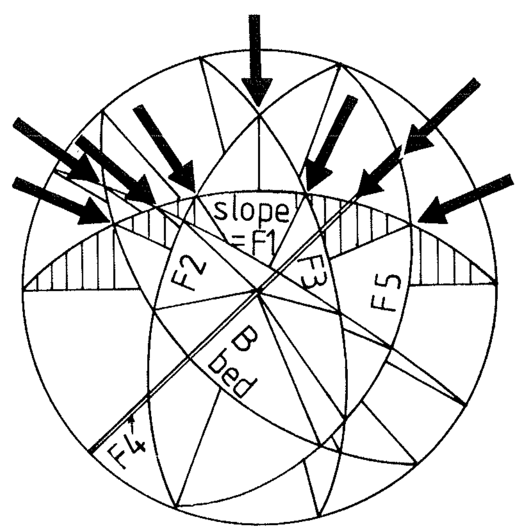


FIGURE 5 Fan of wedges. The slope S = F1 is controlled by fracture F1 and is therefore structural. The numerous structural wedges, represented by arrows, indicate a potential large rock or debris slide.

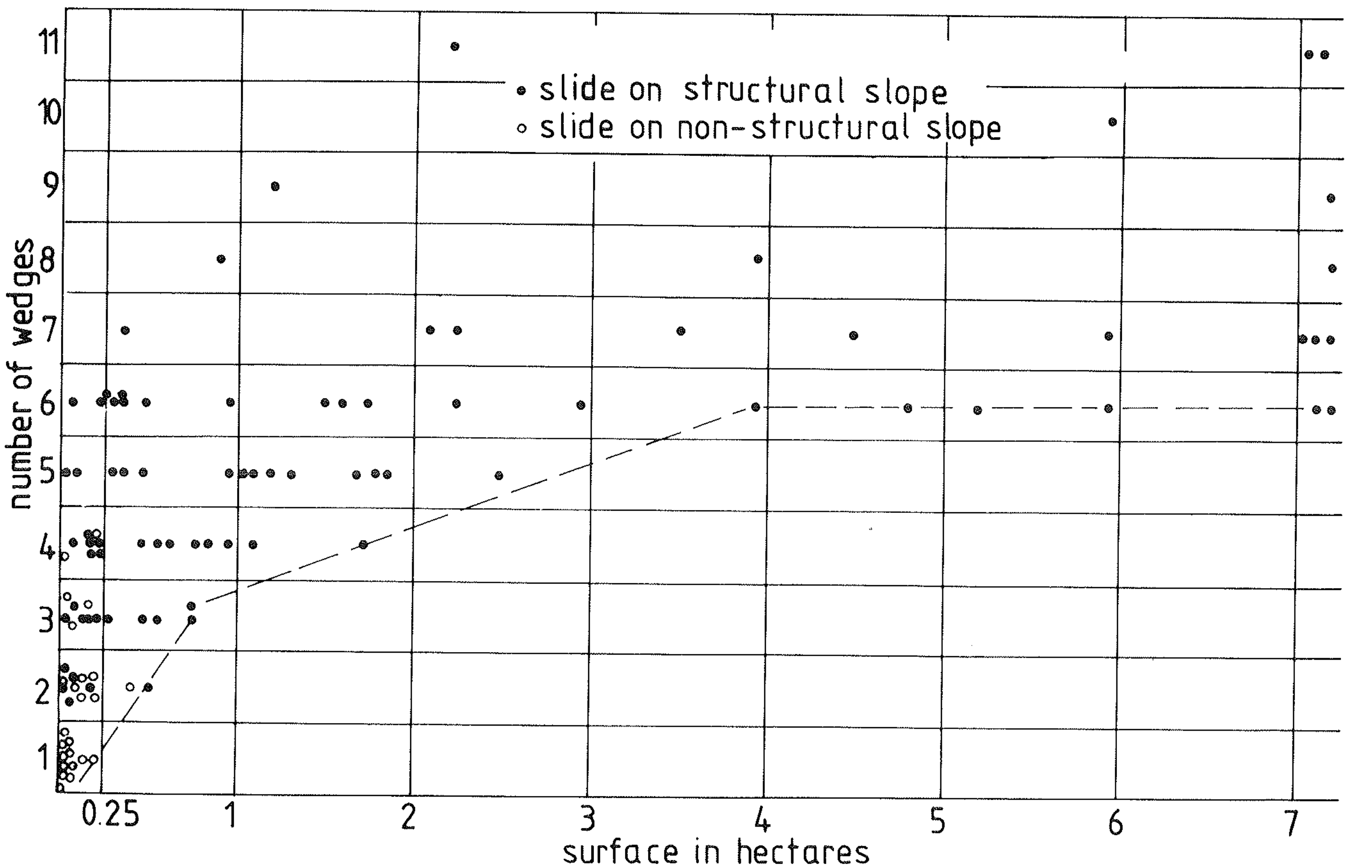


FIGURE 6 Number of structural wedges by surface of rock and debris slides.

- With 4 to 5 wedges, about 70 percent of the slides exceeded 0.25 hectare, provided a structural slope was present.
- With 6 to 7 wedges, 96 percent of the slides exceeded 0.25 hectare, 70 percent exceeded 1.0 hectare, and 42 percent had surface areas greater than 3.0 hectares.

Another working hypothesis was that the surface areas of slides tend to increase with the presence of a structural slope. The statistics fully confirmed this hypothesis, as follows (also see Figure 6):

- The surface area of slides is never very large on non-structural slopes. Fully 75 percent of the slides had surface areas less than 1600 m^2, whereas none exceeded 45 ha (one hectare equals 10 000 m^2).
- On structural slopes, about 65 percent of the slides exceeded 0.25 ha in surface area. It is shown in Figure 6 that the surface areas of slides on structural slopes clearly tend to increase as the number of wedges increases.
- The statistics also demonstrate that fracture planes are as prone as bedding planes to create slides. In fact, 67 percent of the slides presenting a planar structural component occurred on fracture planes.

Incline of Slope

Seventy-seven percent of the slides occurred on slopes with an incline ranging from 40 to 60 grads, whereas only 7 percent took place on slopes with an incline ranging between 35 and 40 grads. Rocks are frequently not present below 30 to 35 grads; the slopes are usually covered with an eluvial or colluvial mantel of varying thickness.

Density of Geologic Planes

The density of geologic planes is difficult to estimate. The assignment of numerical values for density becomes even more complex for cases in which several planes exist. Reliable estimations can be made, however, by taking into account the continuity of each of these planes through the space and density of each.

One should also take into account the interval between two geologic planes of the same type. Experience showed that intervals (i.e., density of a fracturation) vary considerably within a given area. Moreover, the study reveals that large slides often take place in areas of moderate density. As a variable for predicting the potential risks of landslides, the density of geologic planes proved insignificant and was therefore abandoned.

The method should nevertheless be used for estimating the stability of given spots that are restricted to areas of limited size. In these cases, the coating and filling between fractures and the quality of their surfaces must also be considered.

Type of Rock and Its State of Weathering

Weathering, which is a principal cause of slides, depends mainly on the lithological nature of the mother-rock. Theoretically, the type of rocks least subject to weathering are those constituted by hard minerals that are nonreactive to acids, such as quartzite. Marl, calcschist, and alternating layers of rock of a clay origin mixed with carbonate rock are among those most prone to weathering.

When slide surface area was compared with the type of rock, a statistical analysis along roads of the Kathmandu syncline over sections totaling 110 km allowed the compilation of a lithological coefficient for the potential to slide (see Table 1). Experience showed that the coefficient should have been somewhat lower when the structural slope was created by a fracture system through schists or clay and marly rocks interbedded with hard rocks like quartzite, dolomite quartzite, and gneiss.

Minerals

The lithological coefficient for potential to slide can be increased or decreased according to the presence or absence of minerals subject to weathering. In general, the coefficient should be increased when pyrite, sericite, and graphite are present. The weathering of pyrite, in which sulfuric acid is given off, can considerably weaken rocks, even quartzite. Sericite, graphite, and, in some instances, micas, when oriented according to the slide plane, increase the slippage of rock. Carbonaceous matter, which often contains pyrite, is able to increase the weathering of rock for the reason just mentioned. Chlorite acts through oxidation to also weaken the rock.

Water, Springs, and Seepages

The presence of water, in the form of rivulets adjacent to weathered rock, or springs and seepages within weathered rock, obviously increases the potential occurrence of a slide. The quantifiable role played by water is nevertheless difficult to analyze because springs and seepages are often intermittent as opposed to perennial, and therefore may not be visible at the time of the surveys. Consequently, only 35 percent of the landslides surveyed appeared to be directly connected with rivulets, springs, and seepages, whereas 17 percent appeared indirectly connected.

Faults and Thrust Faults

The presence of faults or thrust faults within or adjacent to a critical area has a significant influence on rock and soil stability, although it is difficult to quantify. In Nepal, the areas adjacent to vertical faults were often found to be connected to slide areas of the translational and rotational type because of their chronic, activity-producing earthquake seismicity. They were also found to induce deep weathering of the rocks. Thrust faults in Nepal, which generally are older and less active than vertical faults, were found to often induce strong gully erosion within the crushed rock of the over-thrusted limb.

Morphology of the Site

The shape of the slope was found to be significant for assessing the influence of water. Because they act as natural collectors, concave slopes were found to be far more subject to slope failure (about 60 percent) than planar slopes. Very few convex slopes appeared to be subject to translational slides.

LANDSLIDE RISK MAPS

The results outlined earlier enable a quick but relevant assessment of the stability of any rocky or sub-rocky site to be assembled. The assessment system was subsequently standardized and systematically used for a trail bridge survey by the Suspension Bridge Division of Nepal. Slide risk mapping techniques were concurrently utilized along two sections of roads (Prithvi Rajmarga and Muglin-Narajanghat) in Nepal (*1*, *2*). They were then introduced for stability assessments of a

TABLE 1 TENTATIVE LITHOLOGICAL COEFFICIENTS FOR THE POTENTIAL TO SLIDE IN THE KATHMANDU SYNCLINE

TENTATIVE LITHOLOGICAL COEFFICIENT FOR THE POTENTIAL TO SLIDE (KATHMANDU SYNCLINE)		
	Rocks	Coefficients
1)	Slates, phyllites and schists	10
2)	Slates, phyllites and schists closely interbedded resp. with metasandstones, quartzite and gneiss*	10
3)	Slates, phyllites and schists closely interbedded with calc-slate, calcschists, thin layers of limestone, dolomite and dolomitic quartzite*	13
4)	Quartzite	1
5)	Massive gneiss	1,5
6)	Massive limestone, dolomite	1

section of the Lamosangu-Jiri Road in central Nepal, and ultimately along a 230-km corridor of the Rapti roads project.

The system of slide risk mapping is based on a combination of three basic maps: a slope map, a geological map, and a morphostructural map. The superimposition of these three maps yields a landslide risk map. The landslide risk map is completed by notes on specific and questionable areas of the proposed road alignment.

The final goal of these documents and the accompanying report is to give the civil engineers documents and recommendations to enable them to undertake the correct steps to stabilize existing failures, avoid incorrect design in risky areas, and select optimal alignments for roads and other constructions. The risk mapping technique is further illustrated on a map of a section of the Lamosangu-Jiri Road corridor from km 101 to km 103 (see Figure 7).

The Geological Map

The geological map in Figure 7 depicts the nature of the various rocks according to their lithological potential to slide. The dip of each rock is indicated as is its state of weathering or crushing. Tectonic features such as faults, thrust faults, and folds are also depicted. These are carefully surveyed in the field and are supplemented by aerial photographs. Morphological features, including eroded streams and gullies, other eroded or sliding areas, scarps, cliffs, terraces, and landslides close to or within the corridor, are mapped from the field and by a survey of aerial photographs. The location and behavior of springs and seepages are surveyed with particular care to enable general hydrogeological characteristics to be recorded whenever possible. Soils are also represented and are divided into three categories: colluvial, eluvial, and alluvial.

The Slope Map

The slope map, which is expressed in grads (see Figure 7), is derived directly from the topographical map by counting the number of intervals between contour lines contained in a standard unit, the size of which depends on the scale of the topographical map and its accuracy. For inaccurate topographical maps, a systematic measurement of the upward and downward side-slopes of the alignment permits the results to be adjusted to some extent.

Shown in the slope map are preliminary views of potential instabilities in the area because rock and debris slides preferentially occur on predilected inclines ranging from 35 to 55 grads. The map is also used to superimpose slope values on structural projections for given structural areas of the morphostructural map.

The Morphostructural Map

The morphostructural map (see Figure 7) shows the division of the road corridor into slope units. The directions of the slopes are indicated. Broader units, called structural areas, are also outlined.

Experience revealed that it was possible to divide the rocky and sub-rocky terrains into structural areas in which the rock contained similar structural patterns. The structural areas were delimited by overlaying the results of a regular interval field sampling of rock structures onto Schmidt pole nets. The pole nets allowed the design of the representative structural projections of the various structural areas (Figure 8).

As these statistics demonstrate, a systematic measurement of the rock's planar discontinuities (i.e., bedding and fractures) is essential to assess the stability of rocky and sub-rocky slopes. The structural pattern of the rock determines the structural risk, which is a highly important factor for predicting potential rock slides and most debris slides.

Slope units were distinguished from so-called structural slope units, which are potentially controlled by a bedding or a fracture along which a broad planar failure can be set in motion.

The superimposition of the structural system of a given structural area onto the slope map allows structural slopes to be distinguished from nonstrucural slopes. If the natural slope is equal to or steeper than a planar discontinuity (i.e., bedding or fracture given by the Schmidt projection) and dipping in the same direction, the slope is structural. The mean structural projection of the structural area also gives the wedge's direction and the wedge dips. Finally, this superimposition enables the so-called structural risks for plane and wedge failure to be assigned probabilities from the statistical study.

Rock and Debris Slides Risk Maps

The risk for these types of slides is obtained by superimposing the three maps described earlier (see Figure 7). Tentative proportional weights are attributed to the different lithological groups, hydrological factors, state of weathering and crushing of rock, and tectonics (see Table 2).

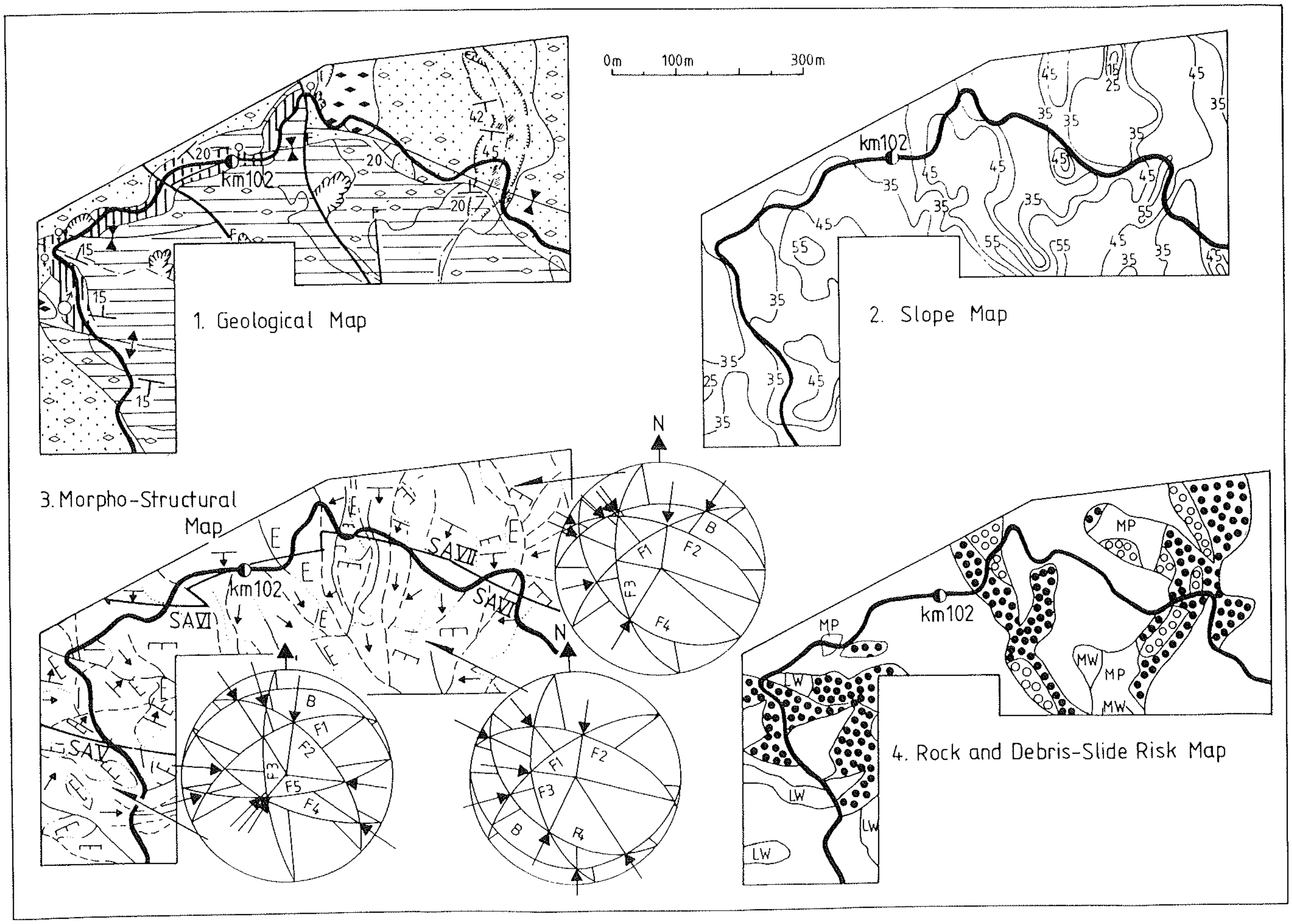
The compilation of field observations, corroborated by further experience, showed that the weight of the structural factor is an important determinant of the site's degree of susceptibility to and the potential size of a rock failure. Wedge failures were found to not exceed 0.25 hectare in size, whereas planar failures in combination with wedge patterns were sometimes found to reach very large proportions.

Two main categories of structural risk were consequently determined, to which were added the other factors mentioned earlier for rock and debris failure. The first category is the risk of plane failure; the second is the risk of wedge failure. According to the type of potential slide plane (bedding or fracture), its dip, the wedge pattern accompanying it, and the other slide factors, three degrees of potential plane failure were determined: low, medium, and high. Three degrees of potential wedge failure were determined in the same manner for wedge patterns in the absence of slide planes.

The probabilities of potential rock and debris slides given on the map approximate the degree of risk existing within the natural slopes. The units represented on the map show general areas of risk. Specific spots within these units, instead of the whole unit, can be affected by the specified risk.

Cut-slopes can create plane or wedge failures when they cross low- or medium-risk areas. Such cases especially occur in 30- to 35-grad slopes, which tend to retain water and consequently lead to deeper weathering of rock. Nevertheless, large failures along cut-slopes usually appear within naturally high-risk areas.

Structures within intensely folded areas are atypical. Statements on risk within these areas are given both on the map and in the accompanying notes. Tectonized areas are also indicated



1. Geological Map

- Thick eluvial or colluvial soil
- Rather thin eluvial or colluvial soil
  In general sparse outcrops of rock
- Calcareous quartzite with laminae of phyllite; lithological susceptibility of sliding very high
- Carbonaceous, micaceous, and garnet phyllite; lithological susceptibility of sliding very high to high
- F Fault
- Anticline and syncline axis
- 42 Dip of the rock
- Crushed rock
- Spring and seepage
- Landslide
- Groundwater level
- Unstable area

2. Slope Map

- 45 Slope contour line (grads)

3. Morphostructural Map

- Ridge or crest
- Sharp ridge or crest
- Rivulet
- Limit of slope unit
- Nonstructural slope unit
- Possible structural slope unit (bed of rock)
- Possible structural slope unit (fracture)

4. Rock and Debris Slide Risk Map

Risk of large failures

- High risk of planar failure
- MP Medium risk of planar failure
- LP Low risk of planar failure

Risk of medium and small failures

- High risk of wedge failure
- MW Medium risk of wedge failure
- LW Low risk of wedge failure
- Very low risk of rock and debris slides; possible soil failure within wet areas

FIGURE 7 Examples of geological, morphostructural, and slope maps, and resulting risk map from km 101 to 103 in the Lamosangu-Jiri road corridor.

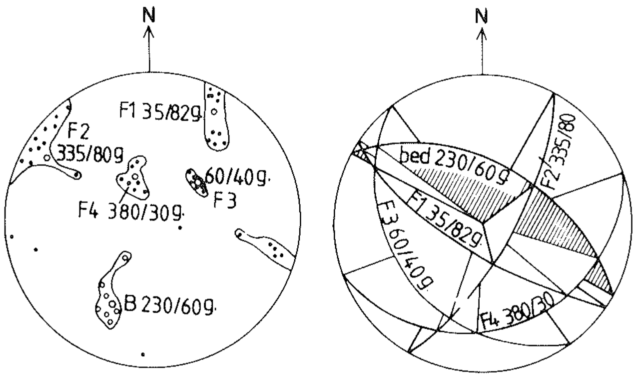


FIGURE 8 Pole net of a structural area and its corresponding structure. The structure at the right is a mean structure that represents a structural area obtained from the pole net. The poles are reported from structures measured in different locations of the structural area.

directly on the landslide risk map. Except along ridges or within rather flat areas, the risk of rock failure in tectonized areas is in principle rather high, even if the structural risk is low.

From Table 2 one can point out the following tentative degrees of specific risks:

- Up to 65 percent there is a low risk of wedge or planar failure,
- From 65 percent to 85 percent there is a moderate risk of wedge or planar failure, and
- Above 85 percent there is a high risk of wedge or planar failure.

Evaluation of the Results

The three high-risk areas visible on the rock and debris slide risk map (Figure 7) were found to be the scene of major slides in 1983 and 1984 after completion of the road. When the first geological reconnaissance was made in 1982, no visible signs were observed, except that the convex morphological aspect of those particular areas and their hydrological and lithological components were found to be prone to slides.

LANDSLIDE RISK MAPPING BY COMPUTER

Few prior examples of computerized landslide risk mapping exist. Those that do exist are based exclusively on the dip of the rock beds or on rain density and earthquake frequency (*9-11*). One interesting method, developed in Italy, is based on the collection of the 20 main attributes of terrain, including lithology, vegetation, elevation, slope, vertical and horizontal slope convexity, and stream rate (*12, 13*). Mathematical models were created from the combination of these attributes. This system nevertheless requires more data and documentation than are generally available in developing countries.

The computerized landslide risk mapping system described in this paper is based on the results, statements, and methods described in the preceding sections. An important concern is to create a system that can be adapted to the needs of civil engineering projects in developing countries. This requires that the system operate with data collected in remote areas with portable instruments, large-scale topographical maps, and

TABLE 2 TENTATIVE WEIGHTS FOR FACTORS THAT LEAD TO ROCK AND DEBRIS SLIDES

TENTATIVE WEIGHTS FOR FACTORS LEADING TO ROCK AND DEBRIS-SLIDES (IN %)	
1a. Structural Risk of Planar Failure ("Plane Risk")	
	Percentage of total weight
Condition A1: The potential plane failure is controlled by the beds of the rock. The slope is therefore structural. Three or more wedges are present, at least one of which is central or centro-lateral. Other are not very lateral. If the dip of the potential bed plane failure is less than 20 grads, the percentage weight is reduced to 20%.	35%
Condition A2: Same as A1, but the slope is controlled by two geologic planes with orientation and dip values approximatel/ those of the slope. Reduced to 20% when the dips of the plane are less than 20 grads.	40%
Condition A3: Same as A1, but the slope is controlled by a fracture. Reduced to 20% when the plane dip is less than 20 grads.	30%
Condition A4: Same as A1, but the slope is controlled by a fracture cutting a more or less thick layer of sandstone, quartzite, gneiss, limestone or dolomite interbedded with rock of clay origin. Reduced to 15% when the plane dip is less than 20 grads.	25%
Condition B: The potential plane failure is controlled by the bed or a fracture of the rock as in condition A, but only one wedge is central or centro-lateral and a total of only two wedges are present within the ce ral and lateral sector. Reduced to 12% when the plane dip is less than 20 grads.	20%
Condition C: The potential plane failure is controlled by the bed or a fracture of the rock as in condition, A and B but there are no central or lateral wedges. Only very lateral wedges are present. Reduced to 6% when the plane dip is less than 20 grads.	12%
1b. Structural Risk of Wedge Failure ("Wedge Risk")	
15% of the total is added for each central or lateral wedge (up to 40 grads from the direction of the slope). When the dip of wedge line is less than 15 grads, the wedge risk is nil.	15% per wedge
2. Lithological Risk	
Lithological Risk should be established for each specific region according to the visible rock failures within given types of rock or group of rocks. The following scale was established for the landslide risk map of the Rapti Road Assessment Project, a 230 km long road Project in Western Nepal. The scale has been simplified for adaptation to other situations.	
a. Fissile green-light brown phyllitic slate with fine laminae and marl beds. Black shales.	20%
b. Sandstone with green and red shale. Black shales with ferruginous quartzite and silty limestone. Light brown laminated slate with calcareous slate. Green phyllite with fine laminae.	15%
c. Gray clay slate with quartzite. White pink quartzite and red purple shale. The percentage is raised 15% when the slope is structural with the bed of rock.	10%
d. Dolomite and limestone. Quartzite.	5%
3. Hydrogeological and Hydrological Risk	
Areas with perennial spring(s) or seepage(s), or areas close to stream(s).	25%
Areas with seasonal spring(s) or seepage(s), or close to stream(s).	20%
Areas with only seasonal rain.	15%
4. Tectonic and Weathering Risk	
Crushed or folded rock, rock with open joints, strongly weathered rock adjacent to faults areas.	20%
Slightly weathered to moderately weathered rock. Not tectonized rock.	10%

generally poor supporting documentation. The system can currently be run on a portable, IBM PC-compatible microcomputer. Its purpose is to integrate more and more terrain instability factors onto a grid after having attributed weights to them.

The project will last 3 years and is financed by the Swiss National Fund for Scientific Research, a government agency. The project will integrate data collected in Nepal during low-volume road projects and in Switzerland. It will also test a method for soil slide risk mapping based mainly on field sample soil tests and measurements of soil anisotropy by pocket seismographs. The first results of the project concern only rock and debris slide risk mapping, in particular the structural risk within the same zone of the Lamosangu-Jiri road corridor that was previously described.

Structural Risk

Slope of the Terrain

The slope mapping of the terrain enables the eluvial, colluvial, and morainic terrains to be roughly distinguished from the rocky or sub-rocky areas. Eluvial soils, produced by weathering of rock within equatorial, tropical, and subtropical regions, are mostly present on slopes with an incline of less than 30 grads in which water penetrates deeply. Colluvial and morainic soils also occur on smooth slopes rarely exceeding 30 grads. All these terrains are strongly influenced by the laws of soil mechanics. The terrain above 30 grads is generally rocky, sometimes with a thin eluvial or colluvial mantel. Stability is mostly determined by the laws of rock mechanics. Detailed field and aerial photographic surveys give more information on the limits of the different terrains. A structural risk is theoretically present on slopes steeper than 30 grads.

Computer Inputs for Slope

The following variables must be assigned values for each point on the terrain grid:

X = horizontal coordinate (meters),
Y = vertical coordinate (meters), and
Z = elevation (meters).

After these topographical values are digitalized on the grid, a special program gives the following variables:

D = incline of the slope (grads),
O = orientation of the slope (from 0 to 400 grads),
A = structural area code, and
G = lithogeological code.

See Figure 9.

Structural Areas

Structural areas are areas in which the rock structure remains roughly similar throughout. They are generally outlined by field measurements, aerial photographs, and pole net plottings.

For a slope with a given incline and orientation, values for the following variables must be entered into the program for each structural area:

NP = number of geologic planes (i.e., bed, schistosity, planes, and fractures);
ID = identification of geologic plane (bed, fracture, or plane of schistosity);
OP = orientation of plane (North = 0 grad);
DP = dip of the plane;
NW = number of structural wedges;
OW = orientation of structural wedges (North = 0 grad); and
DW = dip of the wedge.

For example, a structure with five geologic planes is depicted in Figure 10. At this point on the grid, the slope is structural because it coincides with fracture F1. Seven structural wedges characterize the structure at this point on the grid.

NP = 5.

Identification of a plane (Figure 10), example for fracture 1:

ID1 = fracture 1 (220/60 grads) coincides with the slope incline and orientation of the node.
OP1 = 220 grads.
DP1 = 60 grads.
NW = 7.

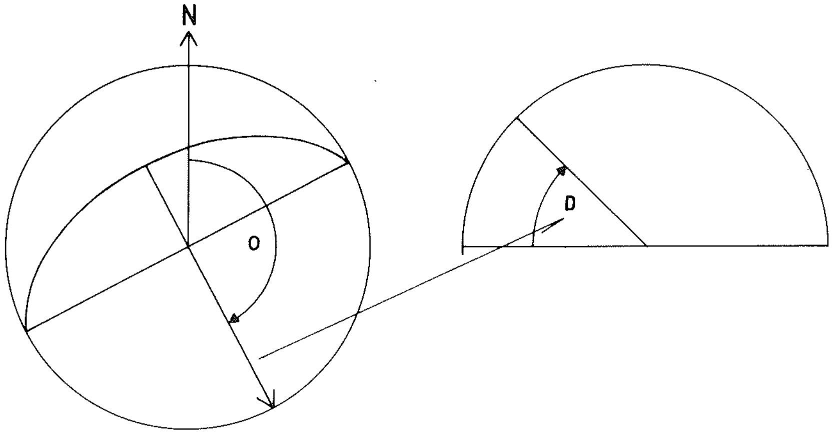


FIGURE 9 Orientation O and dip D of a geologic plane.

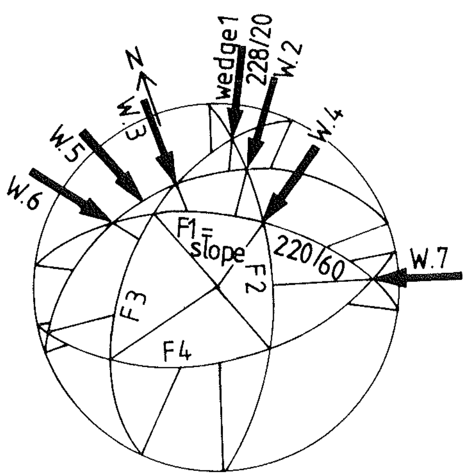


FIGURE 10 Plane and wedge identification.

Identification of a wedge (Figure 10), example for wedge 1:

OW1 = 228 grads.
DW1 = 20 grads.

Parameters Unique to the Method

Values have been attributed to several parameters listed in the following paragraphs by utilizing the results of the statistical study. Several parameters need to be refined through experimentation or research; others are specific for given terrains.

The slope threshold is denoted as RM and is a threshold above which the laws of rock mechanics can be applied to the terrain. The angle has been set to 30 grads for humid and hot climates. Below this value, stability is frequently ruled by the laws of soil mechanics.

The permitted angle, which is denoted as TOP, exists between the slope orientation and the orientation of a given geologic plane, within which the orientation of the plane can be considered equal to that of the slope at the node. This angle depends on the gap values that have been observed within the corresponding structural area. In general, the permitted angle can be set at 30 grads (Figure 11).

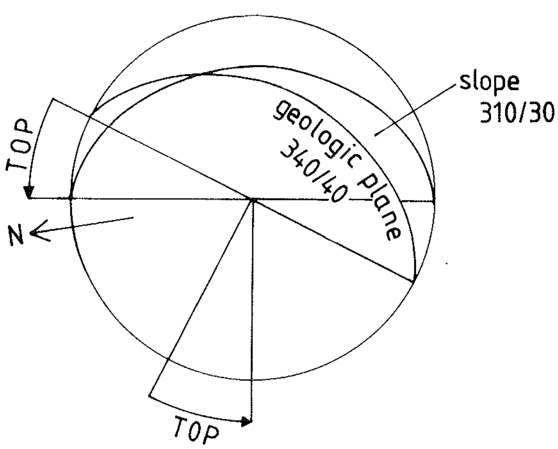


FIGURE 11 Permitted angle TOP between the orientation of a geologic plane and the orientation of a slope.

The permitted angle between the dip of a given geologic plane at the node and the associated slope of the node is denoted as TP. When the dip of the geologic plane is higher than the associated slope of the node (Figure 12), no freedom of movement exists, and, consequently, there is no possibility of a planar slide. Nevertheless, a permitted angle TP is attributed following examination of the variability of the given geologic plane dip within the given structural area. When the dip of the geological plane is less than or equal to the incline of the associated slope, freedom of movement and risk of planar slides exist.

The limit orientation angle of a central sector is denoted as ±OT1. A given sector can be defined as central in regard to the geologic plane under consideration. A central sector therefore contains central wedges (see plane risk A, Figure 13). Such central wedges act as the keystones of a structure. When one central wedge is set in motion, the lateral wedges follow. The risk is considered to be at plane risk A when a central wedge exists.

The statistics presented earlier in this paper indicate that the plane risk decreases when the orientation of the wedges is outside the central sector (i.e., ±OT1 = ±35 grads); the risk in such a case is referred to as plane risk B or C (see Figure 13).

The permitted angle between the slope orientation of a node and the slope orientation of the wedges under consideration is denoted as ±TOW.

When a plane risk A, B, or C is absent, a possible wedge risk still exists. A wedge risk is the risk of a slide along the intersection of intersecting planes. Slides that originate in wedge patterns are comparatively smaller than slides that originate from plane risk A, B, or possibly C. The sector wedge risk is possible at ±40 grads (±TOW) in regard to the direction of the slope at the node. The intensity of the wedge risk depends on the number of wedges and the dip of each (Figure 14).

The number of structural areas is denoted as NAS. A number is attributed to each structural area. Each structural area is defined by a typical structure.

The minimum dip of a geologic plane is denoted as DL. This dip is associated with the nature of the rock. As was stated earlier, any geologic plane, the dip and orientation of which are

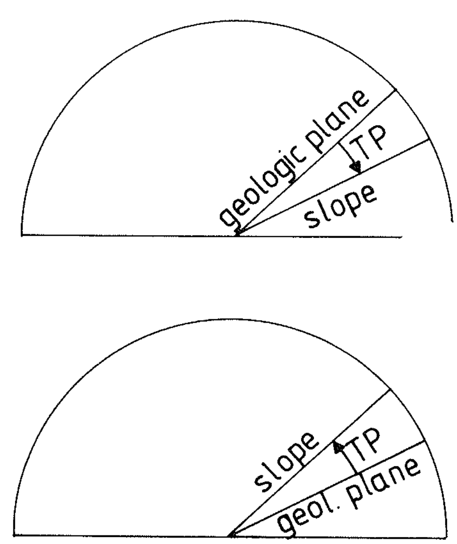
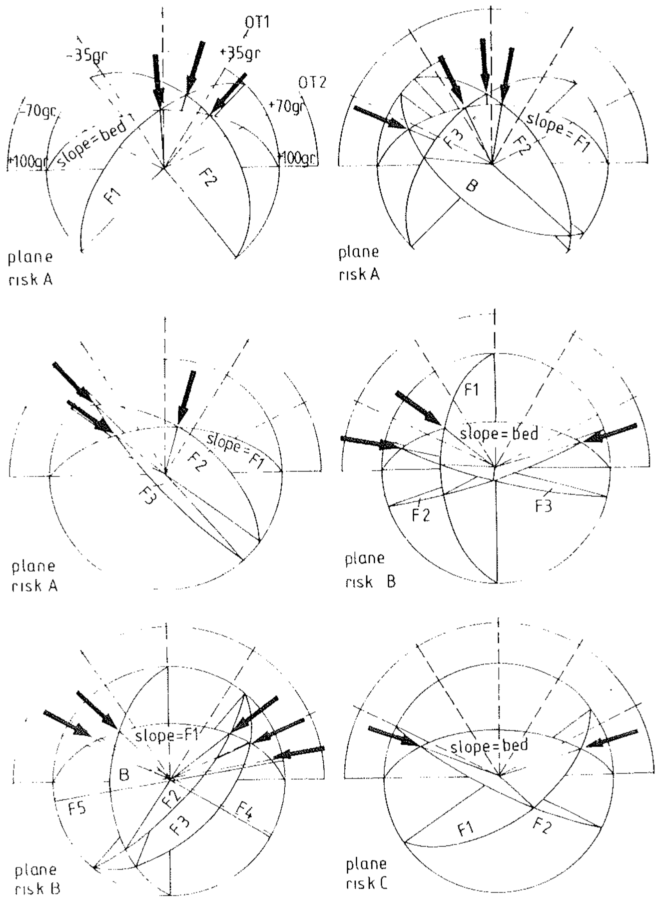


FIGURE 12 Permitted angle TP between a geologic plane and a slope.



From left to right and up to down:

Plane risk A—The risk is high because there are two central wedges and one lateral wedge, and the structural slope is controlled by the beds of the rock.
Plane risk A—A similar situation with four wedges.
Because the slope is controlled by a fracture, the risk is a little less great.
Plane risk A—There is at least one central wedge with two lateral ones; there is also a structural slope controlled by a fracture, but the risk is lower than before.
Plane risk B—There are three wedges and a structural slope controlled by the beds, and three wedges of which none is central.
Plane risk B—Similar to previous one, with five wedges of which none is central. Because the slope is controlled by a fracture and not by the beds, the risk is lower than before.
Plane risk C—Although the slope is controlled by the bed, there are neither central nor lateral wedges. Only very lateral wedges are present.

FIGURE 13 Plane risks A, B, and C.

within the permitted angles TOP and TP, has to be considered a plane of potential failure if the dip is less than or equal to the incline of the slope. In such a case, freedom of movement exists. However, a minimum angle DL exists below which no motion is possible. This angle depends on the friction angle of the rock. The angle is therefore higher for quartzite and limestone than for a clay rock. The minimum angle is also higher along fracture planes than along the beds of a rock.

In addition to the minimum angle DL, local conditions such as intensity of weathering, gaps in joints and fractures, hydrogeology, and tectonics should also be taken into consideration. All data inputs for slope and structural areas are registered on appropriate files on floppy disks.

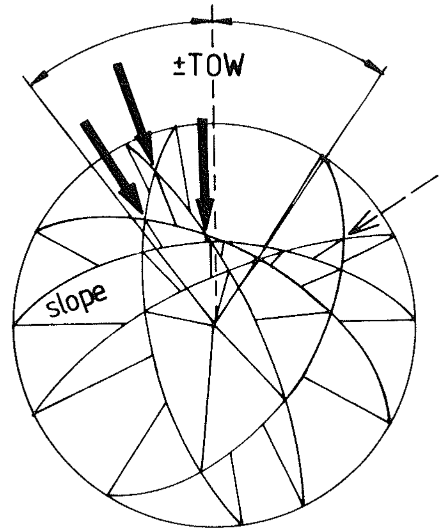


FIGURE 14 Permitted angle TOW between the orientation of a wedge and the orientation of a slope.

Flow Chart of the Computer Processing of Structural Risk

Each node of the grid is considered independently in i rows by j columns.

Step A: For the current node the variables X, Y, Z, D, O, A, and G are read on disk file.
If D < RM, the rock mechanics treatment is feasible; if not, the process will consider a new node and return to step A.
Otherwise, it will search on disk file the specific structural area data corresponding to the considered node ij: NP, ID, OP, DP, NW, and so forth as described earlier.

Step B: Each geologic plane is considered as follows:
If OP is within O ±TOP, the plane is selected; if not, the next plane is considered.
Test for the lithogeology
If DP < DL (G), no slide risk exists; return to step B.
Test for the dipping
If DP > D + TP, no slide risk exists; return to step B.
Test of the number of wedges
The NW wedges of the selected structural area are then considered (see Figure 13).
If OW is within OP ± TO1, NBW (1) is incremented.
If OW is within OP ± TO2, NBW (2) is incremented.
If OW is without OP ± TO2, NBW (3) is incremented.

Terminate step B.
When each geologic plane has been considered, the plane risks RP are established. If no plane has been selected, the process will continue to step C.
If $\Sigma\ NBW = 0$, then $RP_C = 0$.
If $\Sigma\ NBW > 0$, then

1. $NBW(1) + NBW(2) = 0$; $RP_C = 1$.
2. $NBW(1) = 0$; $RP_B = \Sigma\ NBW$.
3. $NBW(1) > 0$; $RP_A = \Sigma\ NBW$.

At this stage, plane risks A, B, and C are registered for the node ij.
Return to step A.

Step C: The wedge risk can be established as follows. The number of wedges *NS* is calculated among all the wedges of the selected structural area if they are within the domain TOW in regard to O and if DW > TDW, after initialization of *RP* and *RW* = 0.
Also, if NS ≥ NW, then RW = 1.

Terminate step C.
Terminate step A, after recording the risk value on a disk file.

When the process has gone through the grid, the computer can map each different risk category (plane risks A, B, and C, and wedge risk) at the proper scale.

The Computer Structural Risk Map

The map shown in Figure 15 is a concatenation of the structural risk (plane risks A, B, C, and wedge risk) in which the black zone corresponds to the major risk (plane risk A). This zone delineates the most dangerous areas for road construction. The dotted areas correspond to minor risks of large failures. The striped areas correspond to wedge risk, or the danger of small to medium failures.

COMPARISON BETWEEN STANDARD RISK MAPPING SYSTEMS AND THE COMPUTER MAPPING SYSTEM

It can first be seen that there is an excellent correlation between both methods. Although the standard map (Figure 7) includes more factors for sliding such as hydrological, hydrogeological, tectonical, and weathering factors, the comparison shows how crucial the structural risk is.

The process that has been developed is therefore appropriate, at least in that geographical area.

The zoning of the computer map shows more accurate limits. In addition, the computer process is able to disclose the risk areas systematically whereas only a general trend can be shown in the standard process.

However, the most relevant improvement of this new method is the usefulness of the systematic aspect of the process, which represents a savings in time in the end. Another significant characteristic of this computer method is the simulation flexibility of the unique parameters according to the observed field data and features, and the local imposed conditions.

This first approach in the computer map allows more confidence to implement the other risk factors and their corresponding weights, provided there is constant control through field experience.

Finally, the implementation of such a simple, facile system will aid the conservation of the road corridor environment in sensitive mountainous subtropical and tropical areas, in regard to both landslides and erosion. It will also help diminish the high maintenance costs of low-volume roads in these regions.

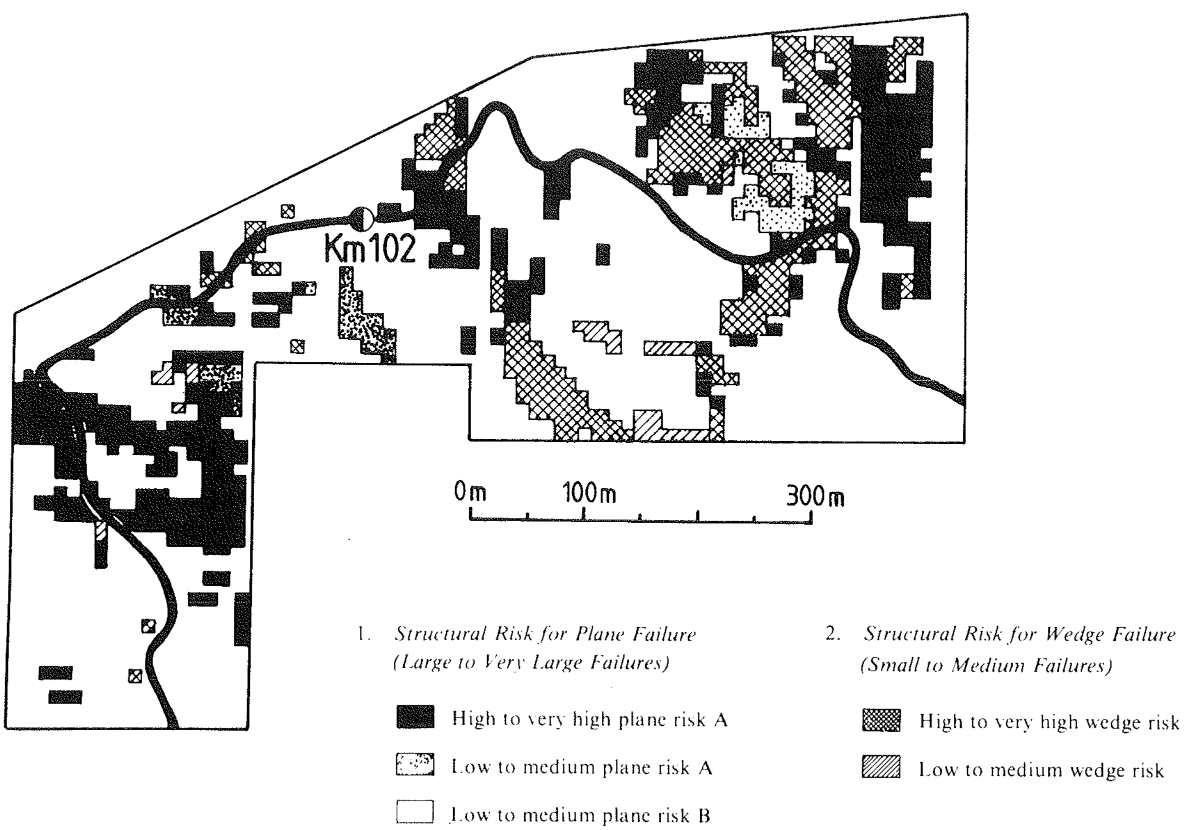


FIGURE 15 Computer structural risk map.

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Efforts To Reduce Construction Costs of Logging Roads in Muskeg and Wet Soils in Southeast Alaska

MELVIN H. DITTMER

A description is provided of the efforts of the U.S. Department of Agriculture, Forest Service, to reduce the costs of low-volume logging roads constructed in the muskeg terrain of southeast Alaska. Muskeg and its characteristics as a road supporting material are described. Techniques to more accurately design and build roads across muskeg and wet soils are discussed. The methods used by the Forest Service to reduce construction costs are analyzed empirically to determine their effectiveness.

An old axiom known to most road engineers is that a requirement for good road construction is "to build and maintain it with a tight roof and a dry basement." That is impossible in southeast Alaska, especially for low-volume and relatively low-cost roads. Roads in this region must be built in the mud and the muskeg, because frequent rainfall prevents the attainment of optimum soil moisture content.

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DEFINITION OF SOUTHEAST ALASKA MUSKEG

Muskeg is defined as terrain composed of a living organic mat of mosses, sedges, or grasses, with or without tree and shrub growth, and underlain by a highly compressible mixture of partially decomposed and disintegrated organic material commonly known as peat or muck (*1*). It is variable in its composition as well as its depth. It may consist of as much as 3,000 percent water; water contents of 900 percent are common in southeast Alaska (*1, 2*). Because summer temperatures are cool (55° F average) and annual rainfalls are high (60 to over 200 in), vegetation in southeast Alaskan muskegs decomposes very slowly (*3*). Some woody material in muskegs is over 8,000 years old (*4*). Muskegs are underlain by material that ranges from solid rock to consolidated silts.

The surface mat of muskeg is frequently interlaced with tree roots. Sometimes these roots are in layers several feet deep that developed over the centuries through the growth cycles of trees and grass that dominated other vegetation (see Figure 1). At other locations, the roots appear to have developed from continuous tree growth, because the roots are dispersed throughout the upper muskeg, as shown in Figure 2.

The top roots in the muskeg may be the extensions of living trees that are growing 100 or more feet away. Although the lower roots in the muskeg are no longer tied to living trees, they retain much of their original tensile strength because they decompose very slowly. The lower portions of many deeper muskegs consist almost entirely of highly compressible peat with some distinguishable roots and logs. Some muskegs have no tree root layers or definable woody debris at all. All of these variations may occur within short distances of each other.

The interspersion of tree roots in muskegs is not extensively discussed in literature on road construction across muskeg terrain (*1, 5-7*). However, roots appear to play a significant role in supporting a road embankment that is being "floated" across a muskeg. The term "floated" is a local colloquialism that is used to describe a shot-rock embankment that is built on top of the muskeg. The support for the embankment is provided by peat, muck, roots, and other compressible and displaceable materials.

FIGURE 1 Muskeg with three tree root layers exposed in road cut.

FIGURE 2 Tree roots exposed throughout muskeg excavation.

Another characteristic of muskeg in southeast Alaska is difficult to visualize by those who have not seen it. Although the water table is approximately at the surface of the muskeg, muskegs are frequently located on slopes greater than 20 percent (see Figure 3). This is possible because of the high rainfall, low transpiration rates, and the retardation of water flows by the fibrous, sponge-like material of the muskeg. As a result, muskegs exist from the valley floor to and including the tops of some low mountains. The characteristics of muskegs are also controlled to some extent by how they were formed. Flat muskegs that developed from the infilling of lakes after glaciers receded from southeast Alaska about 10,000 years ago contain less fibrous and woody material, are much weaker, and have more liquid at their bottoms (*4*). Those muskegs that developed on sloping or well-drained surfaces have more fibrous and woody material throughout their depths and therefore provide more support to a road embankment.

FIGURE 3 Deep muskeg on side hill in which small pools of water are visible.

HISTORY OF ROAD CONSTRUCTION IN SOUTHEAST ALASKA

Before logging roads were built, float planes and boats were the primary mode of travel of residents and visitors in southeast Alaska. Road building for log harvest became significant in the 1950s. At first, log harvest equipment and associated administrative traffic almost exclusively used the roads. As the road systems grew and interconnected, and as isolated logging camps grew into communities, people became dependent on roads. Despite this increasing dependency on road transportation, traffic volumes rarely reach 200 vehicles per day (vpd) on the most-used mainline systems. The vast majority of roads that are being constructed today are low-speed, single-lane, logging roads with traffic volumes of less than 50 vpd.

Since the initiation of logging road construction in the 1950s, over 2,000 mi of permanent Forest Service roads have been constructed and are in service on the Tongass National Forest. Many miles of temporary or "logger's choice" roads were also constructed. Until the 1970s, most of the roads were "logger's choice" roads and only the mainline roads were designed by

Forest Service engineers. Many of the early road construction methods that were developed by the loggers and the Forest Service continue to be used today. These methods consist of clearing the roadway of timber and end-dumping shot-rock to a depth needed to support the dump trucks. Although off-highway logging trucks use the road, the critical weight vehicle, in terms of axle loading and road degradation, is the loaded dump truck, which weights up to 90 kips.

Nearly all of the terrain crossed by the logging roads is covered by vegetative matter to various depths. In regions other than muskegs, the typical ground profile consists of up to 4 ft of decaying organic material that is underlain by anything from weak, saturated soils to solid rock.

In most regions, solid rock is readily available and has been used as an embankment material to build across muskegs and over soft, wet soils. Quarries are commonly located adjacent to the road and are less than 2 mi apart. Shot-rock embankments that are placed on the road vary from about 1 ft in depth, to documented depths of 12 ft, and estimated depths of 20 ft or more in deep, weak muskegs (*8*). Of course, these deep muskegs are avoided if possible.

PROBLEMS AND SOLUTIONS FOR ROAD CONSTRUCTION IN MUSKEG TERRAIN

Over the past 20 years, road builders and Forest Service engineers in southeast Alaska have developed basic guidelines and criteria for designing and building logging roads in muskeg regions. These are described in the following sections.

Guidelines

One of the important instruments used in field investigation is the metal rod probe. The probe is 1/2 in in diameter and has a blunt point. It is pushed through the muskeg to determine its depth and obtain information about the underlying material (soil, gravel, or solid rock). Some road designers are recording the ease or difficulty of pushing the probe into the muskeg as an indicator of the fill embankment needed. The probe is sometimes used the entire length of the planned road even in timbered regions in which shallow or no muskegs exist. In regions in which no muskegs are found, information is recorded on the probe depth, the difficulty to probe, and sometimes the underlying material. Probes are also made through muskegs approximately 20 ft on each side of the planned road centerline to obtain a profile of the terrain beneath the muskeg.

The compacted depth of shot-rock needed to support the heavy truck loads over the soft, wet mineral soils (other than muskeg) in southeast Alaska is 2.0 to 2.5 ft. The depth of shot-rock needed over muskegs varies from 3.5 ft to two-thirds the depth of the muskeg, but averages about 5.0 ft.

Shot-rock embankment requirements for a road with 14-ft top width (including turnouts) is about 11,500 to 14,000 yd^3/mi. Roads that have a 16-ft top width are usually built for faster traffic and require between 13,000 and 17,000 yd^3 of rock embankment per mile. Approximately 70 percent of road construction costs are associated with obtaining, hauling, and placing the shot-rock.

When rock embankments are placed on muskegs with root mats in the upper horizons, every attempt should be made to load the mat uniformly. Live loads should not be concentrated on the embankment until it is at least 3 ft thick. The objective is to avoid puncturing the root mat that directly supports the embankment. It takes at least 3 ft of uncompacted shot-rock to distribute the live vehicle loads uniformly on the underlying muskeg. The least combination of live and dead loading per ft^2 (about 1,000 psf) occurs with a rock depth of about 4.5 ft.

Typical Design Criteria

The following key design criteria are provided to familiarize the reader with the limitations and costs of logging road construction in muskegs.

The top width of roads is 14 to 16 ft in addition to 10-ft-wide turnouts that are located approximately 500 ft apart. Traffic design volumes are 0 to 100 vpd. Profile grades follow the terrain as much as possible and may approach 20 percent. The road alignment follows the terrain as side-slopes steepen. Design speeds are 5 to 30 mph. Construction costs usually control over haul costs and road maintenance in the economic equation.

The average road cost of a Forest Service public works contract in southeast Alaska in 1985 was approximately \$167,000/mi, not including bridges. The costs can be broken down as follows:

	Costs (\$)	*(%)*
Clearing and grubbing	17,000	10
Roadway excavation	19,000	11
Shot-rock embankment	116,000	70
Small drainage, seeding and mobilization	15,000	9
Total, not including bridges	167,000	100

It should be noted that bridge costs vary widely according to the drainages crossed, but add an average of about \$35,000/mi to the cost of roads.

The average depth of a shot-rock embankment is 5.0 ft in muskegs and 2.5 ft in regions other than muskegs. About 15 percent of the average road length is located in muskegs. The average mile of road uses 13,500 yd^3 (22,000 tons) of crushed or shot-rock embankment.

As can be gathered from this summary of information, the greatest opportunity to reduce road construction costs appears to be in shot-rock embankment. This is also the category in which the Forest Service has concentrated its efforts to reduce costs.

REDUCING THE COST OF SHOT-ROCK EMBANKMENTS

Application of Geotextiles

The first scientific approach toward reducing the amount of shot-rock embankment by the Forest Service in Alaska was made in 1975 by then regional geotechnical engineer William Vischer (*2*). A geotextile was placed over a muskeg surface and lifts of shot-rock were placed over the geotextile in varying thicknesses. A typical logging road was selected 20 mi south of Petersburg, Alaska. A test section of muskeg 8 to 11 ft deep and

700 ft long was selected. The muskeg was probed to determine its depths and a vane shear apparatus was used to determine the apparent shear strength of the in-place (peat) muskeg. The shear strengths ranged from 50 to 350 psf. The average was about 200 psf. The water content of the muskeg was about 960 percent.

A nonwoven polypropylene material called fibretex that weighed 420 gm/m^2 was used as a test fabric to cover the muskeg section. It had a strip tensile strength of about 85 lb/in. It was tested in both single and double layers. The instrumentation used by Vischer included strain gauges and settlement plates.

Cores were also drilled after the road was completed to confirm thickness data determined by volumes placed. The size of shot-rock was highly variable, and ranged from sizes of 4 ft across to gravel and sand sizes.

Vischer concluded that a savings of 28 percent in shot-rock was obtained by using fabric matting across weak muskegs. Little or no advantage was gained by placing two layers of fabric. Negligible benefits were derived from fabrics that were placed on firm muskegs. Fabrics acted only as flexible tensile reinforcing members that distributed the load over the entire subgrade width.

Vischer developed a thickness design guide for muskeg as a result of his study (*2*). The guide has not been used to its full advantage by the Forest Service because of the considerable amount of site investigation and testing that is required. More experienced field engineers are also concerned that the test was too limited in scope and the guide therefore does not accurately reflect the highly variable nature of muskegs.

Forest Service engineers continued to occasionally use nonwoven fabrics in muskeg but were uncertain as to their value. In 1985 a second project was selected for study but with less pretesting and instrumentation. On the Bohemia Road near Kake, Alaska, 14 sections of muskeg were crossed with fabric for a total road length of 6,704 ft. Nine sections of muskeg were crossed without fabric for a total road length of 5,432 ft (*8*).

An effort was made to mix the fabric and nonfabric sections so that they shared equal amounts of the variations in muskeg. However, a critique by geotechnical engineers indicated that the project engineer tended to follow engineering rather than scientific procedures. Consequently, fabric was placed more frequently on the weaker muskegs in an effort to reduce the amount of embankment. It was concluded that the amount of embankment that can be reduced with fabric is slightly more than the data that were recorded on this project indicated.

A Typar 3471 fabric was used. Probe tests indicated muskeg depths from about 5 to 17 ft. Vane shear tests varied from 50 to 400 psf. The higher vane shear values were usually obtained in root zones or at the bottoms of the muskeg. The amount of 8-in minus rock placed was carefully monitored by the Forest Service project engineer so the depth could be determined (see Figure 4).

No direct correlation existed between vane shear values and the depth of rock, but some indicators were obtained. It was found that when vane shear values averaged less than 300 psf, the embankment depth averaged 6.7 ft. Vane shear values greater than 300 psf resulted in average embankment depths of 3.4 ft. However, the use of shear strength for design is not conclusive because of the limited test data that were obtained in this study.

The depth of muskeg correlated somewhat to the depth of rock embankment, as might be expected. For example, muskegs

FIGURE 4 Placing 8-in minus rock embankment on geotextile fabric.

under 10 ft deep required an average of 4.0 ft of rock embankment, whereas muskegs deeper than 10 ft required an average of 6.7 ft of rock depth. When the strength was divided by the depth, a better correlation with the embankment depth was obtained.

Extensive areas of muskeg were crossed on this project. Because all rock borrow was weighed, it was relatively easy to monitor how much rock was required to cross given areas. The design generally estimated that subsidence of the embankment through the test sections would be half the muskeg depth. An analysis of the data revealed the following:

- The average design depth of rock in muskeg for all test sections was 5.93 ft,
- The average depth of rock placed in all test sections was 5.23 ft,
- The depth of rock placed in muskeg without fabric was 5.50 ft,
- The depth of rock placed in muskeg with fabric was 4.70 ft, and
- The computed rock embankment savings with fabric was 15 percent.

The design did not provide for the reduction in embankment that might result from the use of fabric. In the test sections in which fabric was not used, an average depth of 0.43 ft less embankment was needed than was predicted in the design. This suggests that the embankment in muskeg may commonly have been overdesigned.

Muskeg failures were indicated by large embankment quantities and frequently by an upthrust of muskeg adjacent to the road. As shown in Figure 5, a deep muskeg on the Bohemia project partially failed during embankment laydown on a 16-ft-wide road. Note the bulge adjacent to the road where muskeg was upwardly displaced. Geotextile fabric was used. The embankment is over 6 ft deep at this location.

A benefit-cost analysis of the Bohemia project indicated that the savings derived from using a thinner 8-in minus rock embankment, including haul cost, was approximately equal to the cost of fabric used. Therefore, no advantage appeared to be gained from use of fabric. If shot-rock had been used as embankment because of its lower production cost, the use of

FIGURE 5 A deep muskeg on the Bohemia project.

fabric would have increased embankment costs by approximately 3 percent. Average rock haul distances on this project were about 1.5 mi longer than those usually experienced in southeast Alaska. If haul distances had been shorter, the economic advantages of fabric would have been further reduced.

The Bohemia test site selections slightly favored the use of fabric on the weaker muskegs, which distorted the 15 percent computed savings to the low side. When these savings are compared with Vischer's predicted rock embankment savings of 28 percent, design projections of a 20 percent reduction in rock embankment appear to be realistic when geotextile fabrics are used on weak muskegs (*2*). Geotextiles are therefore generally economical when the embankment haul distance exceeds 2 mi.

An interesting occurrence was observed on this project. At a test section about 1/2-mi-long near the end of the road, rock borrow was placed over frozen muskeg, which is a rare occurrence because snow normally covers the ground during cold weather. One section of the frozen muskeg was covered with Typar 3471 fabric, a second section was not covered with fabric, and a third section was covered with geogrid geotextile. A 3.5-ft lift of 8-in minus rock was placed. It was assumed that considerable settlement would occur the next work season, when the ground thawed and rock trucks resumed hauling over the road. The road surprisingly settled less than 1 ft into the muskeg, except at one section. This approximately 100-ft section had settled several feet even though it was covered with Typar fabric. The muskeg under this failed section was probably much weaker, although this was not detected by surface observation or from probing records.

Improved Design Data

South Wrangell Road, located near Wrangell, Alaska, was constructed by Forest Service contract in 1983. It was designed as a typical single-lane, shot-rock embankment logging road that alternately crossed muskeg and mineral soil terrain. No geotextile fabrics were used in this project. Pay quantities were based on design quantities unless the contractor could show evidence that actual quantities were at least 10 percent greater than design quantities.

After the road was constructed and accepted as complete by the Forest Service, the contractor requested permission to dig test holes in the road to determine the embankment depth. The test holes were dug to substantiate his claim that actual embankment quantities were at least 10 percent more than design quantities. The Forest Service granted the contractor's request.

On October 2 and 3, 1985, the contractor and his engineering consultant began excavating holes in the road with a tracked excavator. The test hole locations were selected by the contractor. Forest Service engineers measured the depth of the shot-rock embankment. These measurements were made in the presence of the contractor's consultant. The results of this investigation are summarized in Table 1.

The results of this limited investigation not only showed that the contractor could not prove his claim, but that the Forest Service had overdesigned the embankment necessary across muskeg on this project. These limited data substantiated the data collected on the Bohemia study that indicated that less embankment was needed in muskeg than was previously believed. Current Forest Service designs have been upgraded to reflect these reduced embankment needs after the designs are correlated with muskeg probing depths.

Incorporation of Timber Slash on the Subgrade

As recently as the late 1970s, woody debris was traditionally kept out of permanent road embankments. However, as traditional practices were examined and efforts were made to reduce costs, the use of unmerchantable logs as corduroy became acceptable. More recently the placement of any available woody debris over the pioneer grade and under the shot-rock embankment has become commonplace (see Figure 6).

In the years before timber slash was used as corduroy, slash was windrowed adjacent to the road. This was not only unsightly but often required extra clearing to create space for the slash. The use of slash as corduroy has reduced the extra clearing needed for debris storage, which in turn has reduced road construction costs slightly.

TABLE 1 RESULTS OF DEPTH MEASUREMENTS OF SHOT-ROCK EMBANKMENT

Road Number	Station Number	Design Depth (ft)	Actual Depth (ft)	Difference (ft)	In Muskeg
6270	529+30	3.8	2.65	-1.15	yes
6270	485+53	4.2	3.55	-0.65	yes
6270	326+20	7.3	4.10	-3.20	yes
6270	264+80	5.5	4.05	-1.45	yes
6290	864+18	5.1	2.85	-2.25	yes
6290	859+69	2.0	2.50	+0.50	no
6290	734+59	7.6	3.95	-3.65	yes

FIGURE 6 Timber slash placed on the road subgrade as corduroy before placing rock embankment in a terrain other than muskeg.

Because the use of timber slash as an effective corduroy depends on its availability and the skill and commitment of the road contractor, the cost savings are difficult to determine. It has been estimated by experienced Forest Service construction engineers that when debris is uniformly spread and mashed down over the rough subgrade before the shot-rock is placed, a savings of at least 1/2 ft of rock can be realized in terrain other than muskeg. No studies or formal documentation of the estimated savings have been made. This is a subject that could be researched or studied in southeast Alaska. Because little or no slash is available in muskegs, and the end haul of slash is impractical, very little corduroying is done over open muskegs, and then usually only with logs.

Contract language has been developed to ensure that contractors use timber slash to reinforce the road subgrade. Also, timber purchasers are using debris more often on temporary (purchaser-elect) roads as a method of reducing quantities of shot-rock, which gives further credence to the technique.

Use of 8-Inch Minus Aggregate

Shot-rock as a road building material has some undesirable characteristics, depending on the quarry and the drilling or blasting contractor. Some of these undesirable features are as follows:

- Large, angular rocks may tend to cut through the muskeg root mat, resulting in increased embankment quantities.
- The use of shot-rock creates a road material that is difficult to maintain for traffic.
- Large rocks are often pushed off the road running surface during final shaping and are therefore wasted. This also creates the appearance of an overly wide road.
- Shot-rock too large to use is left, and wasted, in the quarry.
- The minimum thickness of embankment needed, in theory at least, is the size of the largest rock placed, which could require thicker embankments than are needed for structural support.

The Forest Service has built several roads using 8-in minus rock with and without fabrics to evaluate its cost-effectiveness. Road builders brought in single-stage jaw crushers to meet the 8-in specification requirement. Most of the roads selected were constructed during 1985. The 8-in minus aggregate has proved to be a much improved road building material for facilitating culvert installation and reducing the work of the spreading equipment.

The running surface of the road is more uniform and easier to maintain. Very little rock is wasted along the road, and it is not necessary to increase fill depths simply to accommodate the rock size.

One of the drawbacks to its use is that larger spaces are required to set up the crusher, conveyor, and stockpiles. Crusher downtime can drastically affect production. Although less 8-in minus rock is wasted in pits and along the side of the road, rock quantities are not significantly reduced. It appears that the ungraded smaller rock may have less of a bridging effect on the muskeg mat than the larger shot-rock, which results in higher point loads on the muskeg and the need for slightly greater depths. Because the 8-in material is not graded, it frequently has characteristics that cause it to compress under truck loads. This results in overly wide roads and greater quantities of rock than are needed for shot-rock.

Processing the 8-in minus aggregate from shot-rock adds about 25 percent to the cost of the embankment. This appears to make the use of 8-in aggregate an uneconomical construction practice on very low-use roads in which the advantages of improved haul and maintenance are not realized.

Decreased Axle Loads

In an effort to reduce live loads on the muskeg during construction, equipment weight restrictions were added to some road construction contracts. The weight limitations were directed toward axle loads, but still allowed off-highway loads on standard, dual rear-axle dump trucks. Contract clauses were developed and incorporated into formal contracts.

This requirement appeared to be effective, especially when 8-in rock was used as embankment. The 8-in rock was more susceptible than shot-rock to embankment compression under high point loads. Fewer failures occurred in muskegs during embankment laydown and during embankment haul when weight restrictions on construction equipment were imposed.

Decreased Road Widths

Because traffic speed is not an important economic consideration for the short feeder roads, it was logical to reduce the road width to a minimum to reduce construction costs. These intermittent-use roads were therefore reduced from a 16- to 14-ft top width; curves or shoulders were not widened. Design computations indicate that a width reduction to 14 ft saves about 1,500 yd^3 of embankment per mile.

It is difficult to build a road as narrow as 14 ft on muskeg and wet, soft soils because the embankment tends to spread in width during construction. The Forest Service has been at least partially successful, however, by starting the road slightly narrower than 14 ft and requiring the use of highway-legal rock trucks for embankment haul and laydown.

Payment for Actual Quantities Used

In an attempt to reduce the amount of embankment needed, the Forest Service awarded some contracts that pay for the actual quantities of road embankment used, as opposed to paying for design quantities. The construction inspector observed the pioneer road subgrade and decreased the depth of shot-rock at sections at which strong subgrade materials were encountered. This method has the effect of reducing the direct cost of embankment materials to the Forest Service. It should also reduce the contractor's risk and therefore result in lower bid prices for embankment.

Payment for actual embankment quantities has been only partially effective in reducing construction costs. Road construction contractors tend to put down more rock embankment than is needed and the construction inspector tends to require a smaller depth than is needed. This has resulted in spot failures along the road prism. Whenever a failure occurs, it usually results in more total rock embankment than if a reasonable rock depth had been placed the first time. The occurrence of spot failures also disrupts the contractor's operations because he must move equipment back and forth to correct failures instead of building the road.

Although data to date are limited, it appears that small savings are being derived from reducing the contractor's risk. Contractors on larger projects have expressed satisfaction with actual quantities measurements.

One of the biggest advantages of this approach has been the collection of embankment quantities data over various segments of road. For example, it was learned that it takes less embankment to cross muskegs and slightly more embankment over terrain other than muskeg than was previously believed. This verified the test data that were collected on the Bohemia and South Wrangell roads. As a result, Forest Service design data will be more accurate in the future, and the amount of contract variations will be reduced.

Greater Construction Tolerances

Another approach to reducing construction costs has been to increase construction tolerances. Because the minimum road that a contractor's construction equipment can build is generally adequate for the subsequent logging and administrative traffic, greater tolerances of road geometry could result in lower bid prices.

No noticeable benefits in cost reduction have been observed yet. For one thing, tolerances are already generous. Furthermore, in cases in which the Forest Service pays for actual quantities, greater tolerances allow the contractor to overbuild a road, which causes construction costs to go up instead of down.

Lower Road Standards

Road standards have been reduced in cases in which travel speeds are not significant compared to construction costs. Steeper road grades, tighter alignment, shorter turn-outs, more rolling or undulating of grades, and less concern for vehicle speed in general have been used in an effort to reduce costs.

Some savings have resulted from this approach. At first it was believed that per-mile savings of over 25 percent were being derived. However, it is now apparent that much of the earlier savings can be attributed to heavy competition in the road construction industry. Bid prices have risen the past 2 years and most earlier savings are depleted.

FINDINGS

Savings of about 20 percent in embankment quantities can be realized by placing geotextile fabrics on the subgrade of logging roads that are constructed over weak muskeg in southeast Alaska. This generally makes the use of fabrics economical in cases in which the embankment haul exceeds 2 mi. The embankment quantities needed to construct roads across muskegs are less than was previously believed, especially when good construction practices, such as reducing live loads and minimizing road widths, are followed.

The use of timber slash from the road clearing as a corduroy to support the rock embankment appears to save up to 6 in of the embankment depth on logging roads in southeast Alaska. No studies have been conducted to verify this hypothesis.

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Liquefaction Risk Microzonation for Low-Volume Road Networks

LAN-YU ZHOU AND KANO UESHITA

The vulnerability of low-volume or low-cost roads to earthquakes in seismically active regions has not been given enough consideration. A simple, empirical approach to create a liquefaction risk map was developed. The proposed approach allows one to assess whether or not the soils at a given site will liquefy, and to determine the probability of liquefaction. The method is based on statistical studies of historical cases in which ground failures by liquefaction were or were not observed. The theoretical basis for this approach consists of a statistical risk model that was developed in 1983. Nine data sets from the Haicheng earthquake in 1975 and the Tangshan earthquake in 1976, and 46 data sets from earthquakes that were reported outside the country were used as input data in the development of the model. The approach was tested on liquefaction microzonations of Tienjin, where liquefaction of a large area occurred, and Sian, where an earthquake was predicted, for the purpose of local road network planning.

The vulnerability of low-volume or low-cost roads to earthquakes in seismically active regions has not been given enough consideration. A convenient method to quantitatively evaluate a site's liquefaction risk is needed in the process of planning and designing local road networks that cover a wide area. A simple, empirical approach to create a liquefaction risk map is represented.

Most current methods employ the peak horizontal accelerations of the ground surface as one of the inputs. However, great uncertainties and inaccuracies can result in the determination of peak horizontal accelerations. Therefore, the earthquake magnitude, M, and the epicenter distance, R, are used in the proposed methodology.

The probability of liquefaction as a quantitative risk index in the microzonation was defined to be a function of M, R, and SPT-values, N, with the consideration of total and effective overburden pressures, σ_v and σ_v', for a given depth point under the ground surface.

The calculation of liquefaction probability is based on a statistical risk model that was developed by the authors in 1983 (*1-4*). Nine case data sets from the Haicheng earthquake in 1975 and the Tangshan earthquake in 1976, and 46-case data sets from earthquakes that were reported outside the country were used as input data in the formulation of the model (*5-8*).

The proposed approach allows one to assess whether or not a given site or region will liquefy, and if liquefaction is possible, to assess the probability that it will occur. The method is based on statistical studies of historical liquefaction-induced failures.

The approach was tested on liquefaction microzonations of Tienjin and Sian, where earthquake-induced liquefaction had occurred or was predicted, for local road planning purposes.

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STATISTICAL MODEL

Consider an independent system with a randomly distributed capacity, C, to be subjected to an independent, randomly distributed demand, D. If the occurrence of liquefaction is considered to be such a system, soil resistance against liquefaction can be regarded as C, and seismic intensity can be regarded as D.

When the distributions of C and D are determined by a study of historical data, it can be assumed that two limits exist, α and β, that correspond to C and D, respectively. Therefore, a rational assumption can also be made that the event will surely occur whenever C is smaller than or equal to α, for $D = \beta$, or whenever D is greater than or equal to β, for $C = \alpha$. Therefore, two mutually exclusive and collectively exhaustive subevents exist, E_1 and E_2 (Figure 1), as follows:

$$E_1 = (C < \alpha \mid D = \beta);\ E_2 = (D > \beta \mid C = \alpha).$$

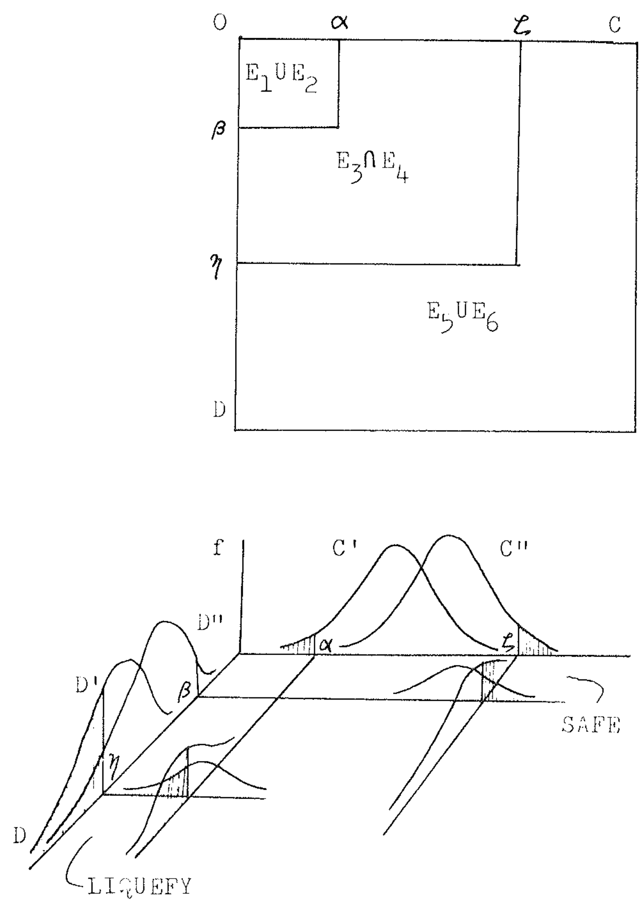

FIGURE 1 Concept of statistical risk model.

The occurrence of liquefaction, $E(L)$, is the union of E_1 and E_2, as follows:

$$E(L) = (E_1 \cup E_2).$$

This means that the soil system will liquefy by the occurrence of either or both E_1 and E_2.

The relation between α and β for certain occurrences of liquefaction has been derived for normal distributions of both C and D, as follows (*1-4*).

$$\alpha = m_c' + (\beta - m_D')\, \sigma_c' / \sigma_D'$$

in which m_c' and σ_c' are the respective mean and standard deviations of C for data sets of liquefaction, and m_D' and σ_D' are those of D.

If it can be similarly assumed that another set of bounds exists, ζ and η, such that liquefaction will not occur whenever C is greater than or equal to ζ, or D is smaller than or equal to η, the same relation between ζ and η for certain nonliquefaction is as follows:

$$\zeta = m_c'' + (\eta - m_D'')\, \sigma_c'' / \sigma_D''$$

in which m_c'' and σ_c'' are the respective mean and standard deviations of C for nonliquefaction data, and m_D'' and σ_D'' are those of D.

This is needed to estimate the probability of liquefaction for a point that falls in the area enclosed by the two boundaries (Figure 2). Consider a given point, $C = C^*$ and $D = D^*$, to be checked in the stochastic domain. According to the earlier equations, the variation limits of C^* and D^* should have the following form:

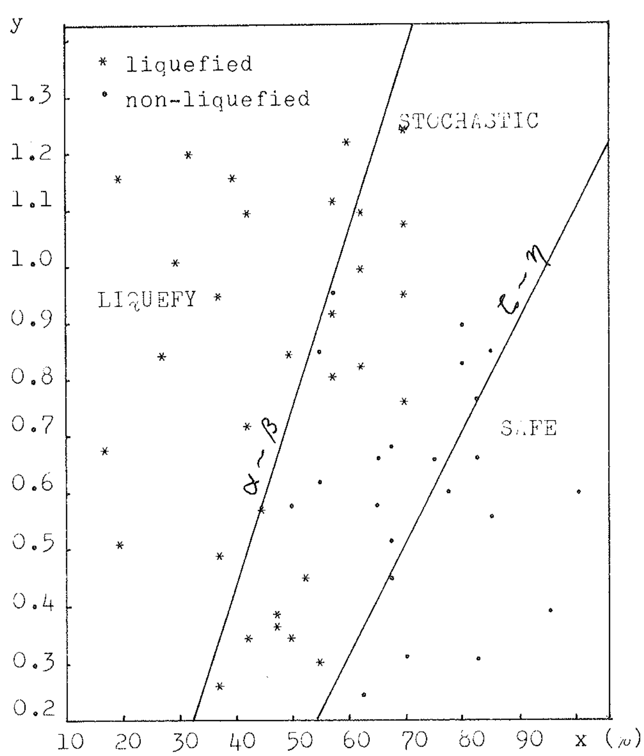

FIGURE 2 Data base and liquefaction limits of SPT-values.

$$\alpha\ (\beta = D^*) \leq C^* \leq \zeta\ (\eta = D^*);\ \eta\ (\zeta = C^*) \leq D^* \leq \beta\ (\alpha = C^*)$$

The probability of the occurrence of the two independent events can be expressed by the following forms, respectively (Figure 3).

$$p(E_3) = \int_{C^*}^{\zeta} f(C \mid D = D^*)\, dC \Big/ \int_{\alpha}^{\zeta} f(C \mid D = D^*)\, dC;\quad p(E_4) = \int_{\eta}^{D^*} f(D \mid C = C^*)\, dD \Big/ \int_{\eta}^{\beta} f(D \mid C = C^*)\, dD$$

Therefore, the probability that the soil system is likely to fail is expressed as follows:

$$p(C^*, D^*) = p(E_3 \cap E_4) \text{ or}$$
$$p(C^*, D^*) = (a - b)(c - d)/(a - c)/(b - d)$$

in which a, b, c, and d are normal cumulative functions of C^* and D^*.

The probability that a given soil layer or an area of ground will wholly liquefy can be expressed as follows, based on Morgan's theory:

$$P_L = 1 - (1 - p_1)(1 - p_2) \ldots\ldots (1 - p_i)$$

in which i is the number of calculated points in a volume of soil.

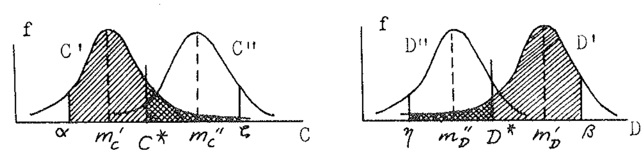

FIGURE 3 Liquefaction probabilities of events E_3 and E_4.

PRELIMINARY MICROZONING IN A REGION

It has been proven that the historical data of both functions, which consist of M and R, and N-values are normal distributions (*1, 4*). The statistics of the historical data sets that were used to formulate the statistical model are listed in the following table.

Parameters	*Liquefied*	*Nonliquefied*
Mean of $x = 210[N/(\sigma_v' + 70)]^{1/2}(\%)$	44.0372	65.3290
Standard deviation of $x(\%)$	11.6451	17.0256
Mean of $y = (M/R)^{1/2}\sigma_v/\sigma_v'$	0.7341	0.6141
Standard deviation of y	0.3916	0.3317

The critical limits of N for liquefaction, Ncr', and for nonliquefaction, Ncr'', have been derived as follows:

$$Ncr' = [(M/R)^{1/2}\sigma_v/\sigma_v' + 0.75]^2(\sigma_v' + 70)/39.69$$

$$Ncr'' = [(M/R)^{1/2}\sigma_v/\sigma_v' + 0.66]^2(\sigma_v' + 70)/17.64$$

where

M is the Richter scale value of the seismic event, R is in km, and σ_v and σ_v' are in kN/m^2.

This procedure, which is outlined in Figure 4, given sufficient consideration of the geological conditions at the site, is applicable to the process of a low-cost road network planning as a tool to determine preliminary microzones within a large area.

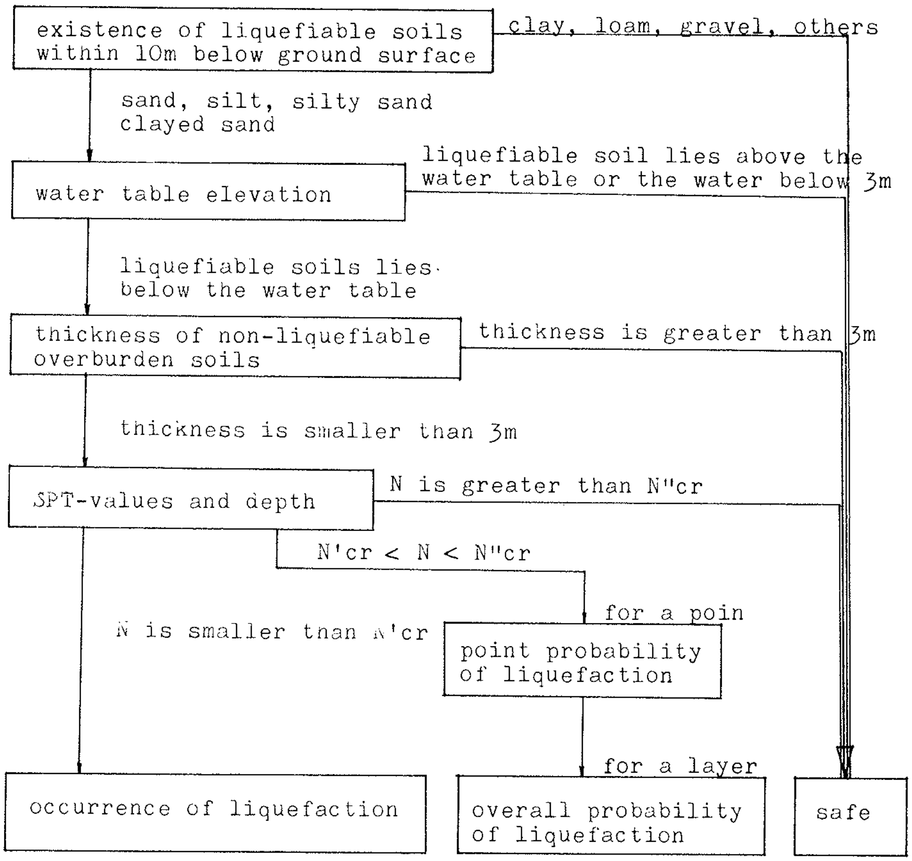

FIGURE 4 Preliminary liquefaction microzonation procedure.

LIQUEFACTION RISK ASSESSMENT IN DETAIL

As was previously mentioned, the liquefaction probability can be evaluated for any given point in the soil and for any given volume or area of a site. The following parameters were obtained in this study:

$$a = \Phi\,(4.048y - 0.087)$$
$$b = \Phi\,(0.085x - 3.782)$$
$$c = \Phi\,(2.554y - 1.874)$$
$$d = \Phi\,(0.050x - 3.557)$$

where

Φ = the standard normal distribution function,
x = $210\,[N/(\sigma_v' + 70)]^{1/2}$, and
y = $(M/R)^{1/2}\sigma_v/\sigma_v'$.

The numerical model mentioned earlier has been programmed to microzone the liquefaction risk in a large region.

Examples

Liquefaction risk microzonation by use of the proposed approach was conducted in an analysis of seismic hazards in the Tangshan-Tienjin region. It was also conducted to check the seismic hazards of a local road network in the Wei River valley (Sian region) where an earthquake of a magnitude greater than 6 has been forecasted.

As shown in Figure 5, the two limits of SPT-values for liquefaction risk analysis vary with the epicenter distance under certain geological conditions in the Tienjin region.

A liquefaction risk microzonation map showing geological boundaries and isograms of *Ncr'* and *Ncr"* was created for the Sian region, as shown in Figure 6. Field investigations were conducted to determine whether or not subsurface conditions conducive to liquefaction existed within 10 m of the ground

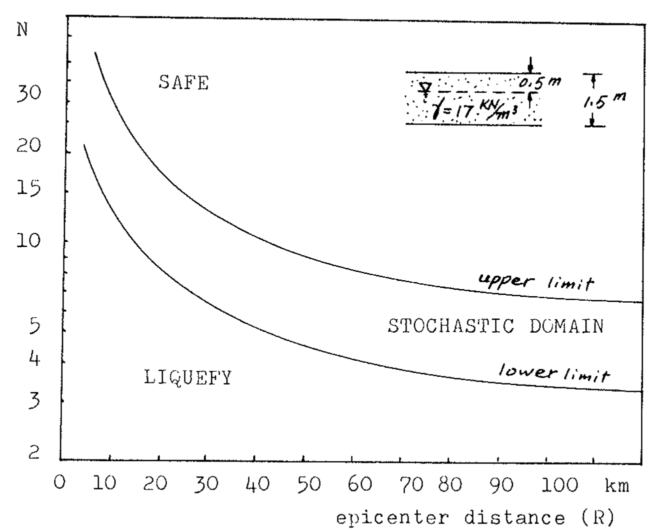

FIGURE 5 Liquefaction limits versus epicenter distance in Tienjin.

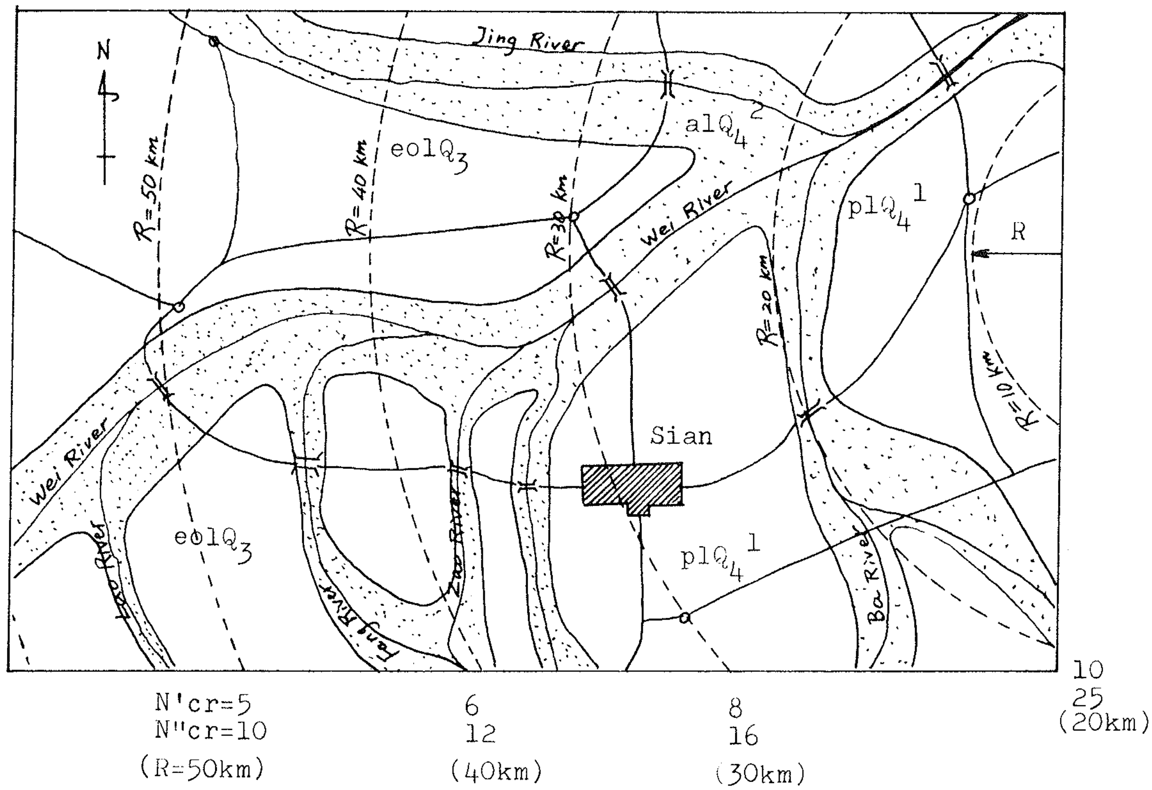

FIGURE 6 Liquefaction risk map of Sian region.

surface in the region to be microzoned. It was found that they were mainly distributed in the soils shown as al Q_4^2 in Figure 6. The second step was to determine if these deposits were below the water table. The SPT-values were also determined. The predicted epicenter is about 30 km from downtown Sian.

CONCLUSIONS

The methodology presented for liquefaction risk microzonation of a given region in which a local road network exists or is planned is based on the statistical risk model. This model was derived from 55 data sets of historical cases. Because of the convenience of this approach, it is applicable to the planning and seismic checking of local low-volume and low-cost roads.

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The Design and Construction of Low-Volume Roads in the Northwestern Sahara

JOSE CARLOS DE O.S. HORTA

Funding for low-volume roads is usually restricted. This is true in the Sahara, the greatest desert in the world. Long distances, scarce population centers, and low traffic levels accentuate the need for low-cost roads in the Sahara. Road engineers in the northwestern Sahara have met the challenge of building good roads with little money by taking advantage of specific features of the desert environment and questioning widely accepted specifications and construction practices. Infrequent rainfall and high evaporation rates represent the major advantage of desert environments for road engineers. Dry soils display high bearing capacities and require light pavement structures. A very wide range of natural materials, including highly plastic clays, can be used as pavement materials. Recommendations are provided for the selection and implementation of base materials in the northwestern Sahara. These recommendations are based on 25 years of experience in the Algerian Sahara and might prove useful for similar environments in other parts of the world. Eolian sand represents the major difficulty facing road engineers in desert environments. It forms dunes and sand seas called ergs that usually have awkward reliefs. Uniformly graded eolian sands display the lowest bearing capacities among the subgrade soils of bituminous paved roads in desert environments. Sand drifts should be controlled to prevent roads from being cut off and to maintain trafficability. Experience gathered in the Sahara and other deserts on sand drift control is reviewed. Design recommendations, simple maintenance structures, and practices are mentioned in connection with this problem. A discussion is also provided of the problems raised by eolian sand in connection with the construction of bituminous wearing courses. Other points discussed are drainage and shallow water tables, the effect of soluble salts on bituminous pavements, and possible savings of compaction water.

The Sahara covers an area of about 9 million km^2. In this huge country with a scarce, scattered population, the mostly very long roads represent considerable investments. Low-cost roads are a main requirement in this region perhaps more than anywhere else. The usually low traffic level of the Sahara does not justify high costs.

This situation is a challenge for road engineers concerned with building good roads in the Sahara. In the northwestern Sahara, the challenge was met by questioning widely accepted specifications and construction practices to take advantage of local materials and the specific features of the desert environment.

The bituminous paved roads that were constructed in the last 25 years, mainly by French engineers in Algeria, add up to a few thousand km and provide favorable evidence of the low-cost road technology developed in the northwestern Sahara. The technology discussed here is believed to apply to the region of the Sahara bounded by the Tropic of Cancer in the south and the Atlas mountains in the north, and extending from the Atlantic coast to Libya.

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THE DESERT ENVIRONMENT

Climate

The desert environment may be defined by the annual mean precipitation. In northwest Africa, the isohyets of 100 or 50 mm are usually considered the boundary of the desert. These lines are close to each other and run along the foot of the Atlas mountain ranges. The very low precipitation allows substantial simplification of drainage. Costly appurtenant works, such as lining of ditches, herringbone and French drains, and pervious subbases that are of great importance in humid environments, are not necessary in deserts.

However, deserts are not simply characterized by the mean annual precipitation. The interannual variability of precipitation increases with a decrease in the average rainfall. In some parts of the desert, some years may be dry and one single shower may reach values as high as the mean yearly rainfall. The monthly precipitation usually falls in one single shower. As a result of concentrated precipitation and lack of vegetation, surface run-off may be as high as 30 to 40 percent and cause flash floods. Episodic surface run-off is difficult to predict.

It follows that it is not possible to do without drainage, even in deserts. Sufficient camber should be given to the pavement and shoulders in order to drain off precipitation rapidly. Run-off should not be obstructed, and culverts should be provided at every small channel or gully. Flash floods are particularly fierce at the foot of mountains; many case histories of road stretches being swept away have been reported. Bridges can be replaced in deserts by low-cost fords. However, both fords and culverts should have appropriate protection against scour.

The direct result of the extremely low yearly precipitation is the scarcity of water. Surface waters and shallow water tables are exceptional in desert environments. Ground water is usually very deep. Soaking of the subgrade does not occur except at a few particular places, such as depressions and basins with shallow water tables. Some of these places can be avoided by the road alignment and, if not, the pavement can be constructed on an embankment of suitable height.

The extremely low rainfall in the Sahara coincides with long periods of sunshine that cause extremely high potential evaporation. Evaporation ranges from 2000 to 6000 mm, which represents about 20 to 500 times the corresponding precipitation. As a result of high potential evaporation, desert soils exhibit very low water content and high strength. High evaporation concentrates salts in water and soil.

The relative humidity of the air in the northwestern Sahara drops to about 20 percent in the hot season and slightly exceeds 50 percent in the cool season. An additional peculiarity of the desert climate is the wide range of variation of temperature between day and night. This has implications on the bituminous binders that should have low thermal susceptibility. The development of thermal cracking has been witnessed in several instances, namely on a soil-bitumen pavement near Tamanrasset.

Vegetation

The scarcity of water and the low air humidity (except along the Atlantic coast) result in a very sparse plant cover of small shrubs that are usually dried out. Weeding and uprooting before earthworks is easily performed in the desert. Shrubs can be removed by hand or mechanically by blading. No topsoil needs to be removed.

Soils

The chemical weathering of rocks is negligible in deserts because of the scarcity of water. Mechanical weathering is caused by the wind and changes in temperature. Wind is the primary erosion and transportation agent in desert environments. It carries fine particles in suspension, and continuously shifts sand by saltation and surface creep.

Desert soils are lithosoils without humic horizon or topsoil. There is no clay neoformation in desert soils, but there is an accumulation of salts, which may have negative and positive effects. Calcium carbonate ($CaCO_3$) in Saharan soils is mainly inherited and formed during the pluvial periods of the Quaternary. Calcium carbonate is actually slightly soluble and requires significant amounts of water to be appreciably mobilized.

Gypsum ($CaSO_4{\cdot}2H_2O$) accumulates in most of the desert soils and is found in a more or less hydrated state called hemihydrate ($CaSO_4{\cdot}O{\cdot}5H_2O$), depending on the air and soil humidity. Gypsum is practically ubiquitous in desert soils. This salt has a relatively low solubility, and certain types of gypsum soils (gypcrete and gypcalcrete) can be used as pavement materials, if drainage is good. However, other soluble salts, namely halite or sodium chloride (NaCl), which is the most widespread, may damage the pavements, as is reported later. The soluble salts accumulate in shallow water tables and undrained basins and form crusts.

Other soils typical of deserts have an eolian origin. The wind may erode existing soils and carry away sand and fines. The remaining soil will show a high concentration of coarse elements on the surface. This type of soil is known as reg or desert pavement.

Eolian sand is present practically everywhere in desert soils. When pure, eolian sand forms dunes and sand seas, or ergs, which take up about 20 percent of the surface of the Sahara. Eolian sand is calibrated by the wind. Its size is a function of the wind velocity. Usual values lie between 0.15 and 0.30 mm. The grading curve of pure eolian sand is almost vertical. Desert soils are usually mixtures of different types of soils with eolian sand. They generally show an oversanded, gap-graded granularity with a characteristic sand hunch.

Terrain Morphology

Desert environments include all the types of relief found in more humid climates. The terrain surface of desert countries is to a great extent inherited from humid Quaternary paleoclimates. The terrain morphology changes very slowly under desert conditions. The wide development of dunes is typical of but not exclusive to deserts. The scarcity of water accounts for the stability of desert morphologies other than dune relief. The cohesion of dry desert soils is high, except for eolian sand. As a result, cuts can be constructed with vertical slopes and earthmoving costs can be decreased. However, eolian sand poses difficult problems for the road engineer. The problem of sand drift control will be discussed later.

The problem of wind erosion or deflation is not as acute as the former in the northwestern Sahara. Strong, sand-free winds might occur in very special conditions and could erode cohesionless sand embankments. Unprotected sand embankments could migrate under the action of wind, but this would not happen to cohesive or coarse materials. The latter represent a suitable protection against deflation when spread over sand slopes and surfaces.

The terrain morphology of the northwestern Sahara is generally tabular. As a result, alignments with small horizontal radii are exceptional and there is great latitude as to the location of the road alignment. This should be put to profitable use to bring the alignment nearer to the available deposits of road-building materials. Materials investigations should be performed sufficiently in advance of the final design toward this end.

EOLIAN SAND

Dunes and Ergs

Eolian sand in deserts forms individual dunes and dune seas, or ergs. The surface of ergs is relatively stable, but mobile dune fields exist in some places that are usually formed by barchans, or crescent dunes. These dunes may attain a height of 10 m and a diameter of 30 to 50 m. The barchans appear where the wind always blows in the same direction, and the wing tips indicate the wind and migration direction. In the course of their migration, barchans can cut off roads (Figure 1) if they are not readily fixed or destroyed.

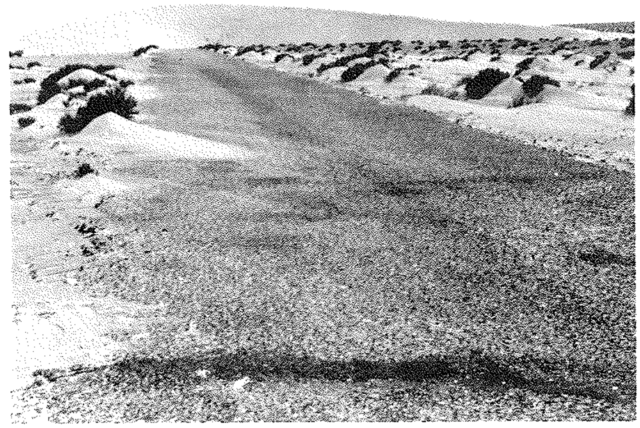

FIGURE 1 A migrating barchan has cut off the road from Laayoune to Bojador, 31.5 km from Laayoune.

Longitudinal undulating dunes appear where the wind blows from more than one direction. They are called siouf, which is the plural of seif. Small dunes form against obstacles. The nebkha, or shrub coppice dune (Figure 2), is smaller than the rebdou. The latter is more than 1 m in height and may grow 3 to 4 m high and 2 to 5 m long. Spaces free of dunes are called sahane if they are equidimensional and feidj if they are elongated like corridors. Many other types of dunes and dune assemblages have been described (*1, 2*).

The surface of the sand seas corresponds to a precarious aerodynamic equilibrium that should not be disturbed by earthworks if the road is to be kept free of sand drifts. However, winds loaded with sand blow everywhere in the desert and problems with sand drifts are not limited to the ergs and dune areas. Because wind energy is incommensurable with human capabilities, sand drift control should be based on the understanding of sand transport by the wind, the knowledge of the particular situations (topography and prevailing winds), and experiments (*3*). The problem of sand drift control should be studied at the design stage to minimize the number of drift-susceptible spots and later the maintenance costs. The techniques of sand drift control and snow drift control are basically the same (*4*). However, unlike snow, sand will not melt in the spring and under certain conditions can stockpile indefinitely.

Rules for Sand Drift Control

In order to avoid sand drift, the design should be based on a few rules. However, some situations will not permit compliance with some of the sand drift control rules; sand control structures and maintenance will have to intervene for these particular road stretches. Neglecting the sand drift control rules could have a significant effect on maintenance costs. The annual maintenance costs would rise and might exceed construction costs, as in the case of a road from the town of Laayaoune to its port.

The following rules should be observed for the location of the road alignment:

- Keep the disturbance of the ground surface to a minimum. The design speed should therefore be decreased to about 75 km/h in the dune areas.
- Preferably locate the road in regions that are free of dunes (sahanes and feidj), and bypass important sand massifs.
- Avoid mobile dune fields.
- Locate the road on regs and coarse sands instead of on surfaces that are covered with eolian sand.
- Avoid crossing dunes and, when unavoidable, select large passes, and locate the road alignment on ground normal to the length of the dune.
- In dune areas, locate the road close to the windward side and far away from the leeward dune slope. If possible, the distance between the road and the dune should be greater than two to three times the dune height.
- Cliffs should be climbed where they are free from sand and exposed to the wind.

The profile of the road should comply with the following rules:

- The gradeline should be raised 0.2 to 0.5 m above the adjacent ground level; soils can be borrowed along the alignment for this purpose.
- Avoid embankments that are higher than 2 to 3 m.
- Avoid cuts; cuts and transitions from cuts to embankments are most susceptible to drifting.

The wind accelerates and sand bounces easily above low embankments and hard pavement surfacing. However, if the embankments are high, the eddies at the windward slope will drop sand on the road. In the past, it was believed that the embankment height should be raised to the crest level of the highest dune, but this resulted in an increase in sand drift, namely in Algeria at National Road 1 between Ghardaia and El Golea.

Trenches and cuts tend to be invaded by sand, especially if they are oblique to the wind direction (Figure 3). Where cuts are unavoidable, their cross-section should be appropriately designed.

Aerodynamic cross-sections should have slopes $H : V \geq 4 : 1$ for embankments and $H : V \geq 6 : 1$ for cuts. Rounding the intersection of the slope with the shoulder would change these limits to 2 : 1 and 4 : 1, respectively. Aerodynamic cross-sections are supposed to be blown clear of sand.

Some authors have stated that cuts deeper than 6 to 8 m are not subject to snow drift (*5*). Deep cuts are usually not necessary in the topography of the northwestern Sahara. Where cuts are unavoidable (cliffs, for instance) the slope may be cut vertically

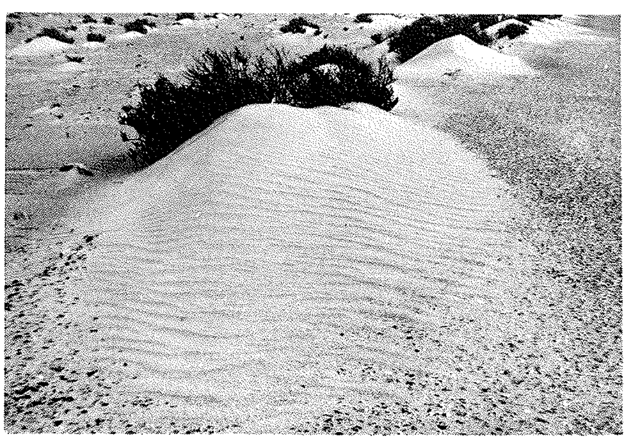

FIGURE 2 Nebkha dunes along the road from Laayoune to Bojador, 130 km to the south of Laayoune.

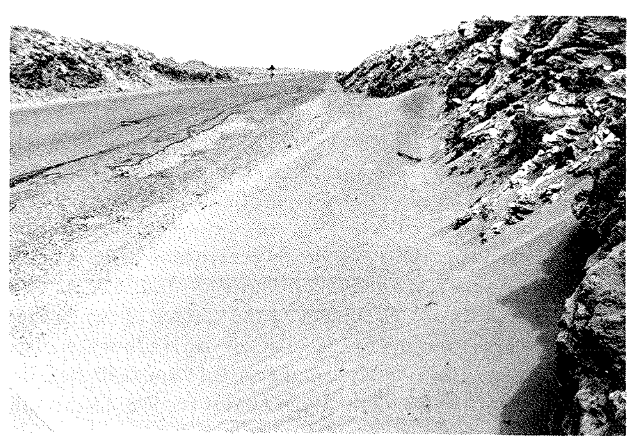

FIGURE 3 Cut section invaded by sand on the road from Tan-Tan to Laayoune, 150 km to the south of Tan-Tan.

and a platform for sand deposition may be provided at the foot of the windward slope. If H is the height of the slope, the width of the platform should be more than $1.2H$. High embankments on curved alignments should not have a downwind camber (*6*).

Another rule in connection with the cross-section is that the width of the road should be increased to a minimum of 10 m in dune areas. A large cross-section will allow traffic to pass, even after sand has drifted over one lane.

Two other sand drift control rules should be implemented during construction. In dune areas, materials should preferably be borrowed from the nearest dunes in such a way as to level them. After construction is completed, a strip 50 m wide along the road should be cleaned and leveled to eliminate any obstacles. If the wind blows from one side only it will be enough to level the windward side. If this rule is ignored, nebkhas and rebdous will invade the road (Figures 2 and 4). As shown in Figure 4, the roadsides have not been cleaned and leveled since completion, and rebdou dunes have formed against obstacles and are now invading the pavement.

Structures and Maintenance Practices to Control Sand Drifts

The maintenance practice for sand drift control comes down to transposing sand with earth-moving equipment. It is very simple, but the costs are high. It requires the use of permanent teams that are ready to intervene after each sandstorm or continuously in areas of migrating dunes. Some methods to stabilize and divert sand are needed to minimize maintenance costs.

Dunes can be stabilized by planting. This is an excellent method, but it requires a water supply. It could be contemplated locally where water is available at a low cost. Spraying the sand surface is another method of stabilization. Salt water can be used to form salt crusts. Bitumen emulsions would form a brittle, thin crust that would not bond with the underlying sand and that could be broken by animals and vehicles. Undermining and exposure of sand would result. Some other types of binders penetrate deeper, build flexible and self-healing protection, and allow vegetation to grow. Stabilization by spraying is expensive. It decreases the roughness of the surface, accelerates migration, and hinders the deposition of sand. Paving with gravel, stones, and cohesive soils is equivalent but cheaper.

The usual methods that are employed to stop sand are trenching, fencing, and panelling. Pannels can also be used to increase the wind velocity and divert sand, but after some time panels usually become covered by sand. However, they can allow maintenance to be programmed independently of sand storms. Trenches should also be cut periodically. Fences are relatively cheap and can be built with local materials, such as palm fronds (Figure 5). Kerr and Nigra recommended a three-fence system to guard against sand drifting (*6*).

The above-mentioned methods and structures can also serve to destroy migrating dunes when they are far enough from the road to be protected. Trenching disrupts the dune temporarily. Movable panels lead to the same result, but the panels must be watched and adjusted to avoid toppling (*7*). Differential stabilization of the dune surface by spraying or paving is the most effective procedure to scatter migrating dunes (*6*).

SUBGRADE SOILS

All the types of subgrade soils that are found in humid climates are also found in desert environments. Calcareous soils and gypsum soils, namely calcrete gravels (GE) and sands (SE) and gypsum sands (SY) and silts (MY), are very widespread in the northwestern Sahara (*8*). Chemical tests are required for the geotechnical identification of soils in this region. The carbonate content should be determined by reaction with hydrochloric acid and expressed as calcium carbonate content (calcium carbonate equivalent). The soluble sulphates content should be determined by reaction with barium chloride and expressed as gypsum content. These identification tests are run together with the Atterberg limits tests on the fraction passing the No. 40 sieve.

The main property of desert subgrade soils is the low water content. The natural water content is a function of the soil type. It is practically nil for cohesionless sands and gravels, around 2 percent for the most common soils, and up to 5 percent for high-plasticity clays (CH). With the exception of sections with water tables shallower than 7.5 m in some depressions and basins, the desert subgrade soils are practically dry and display high California bearing ratios (CBRs). The desert subgrade soils can be grouped in two classes: those with bearing ratios in excess of 20 percent, and eolian sand or uniform poorly graded sand. It is clear that the CBR of Saharan subgrade soils should not be determined on laboratory specimens after soaking.

FIGURE 4 Road from Tan-Tan to Laayoune, 132 km to the south of Tan-Tan.

FIGURE 5 Fences of palm fronds protecting gardens from sand drifts in the oases of In Salah.

Some authors recommend that the CBR be determined at the optimum moisture content (*9*). However, this still appears to be unrealistic in regard to Saharan climatic conditions. Actually, after compaction at the optimum moisture content, the subgrade dries and its bearing capacity increases. Hunt investigated water contents beneath two existing paved desert roads in Libya (*9*). The values found were considerably less than optimum, namely less than 1 percent in the region of Tripoli, and between 2 and 4 percent in the region of Kufra.

Design bearing ratios for desert subgrade soils can either be determined on laboratory specimens molded at low water contents or on specimens molded at the optimum water content and then dried. For this purpose, the drying time is usually 48 hrs, either in the ambient atmosphere or in an oven at 55 to 60°C.

As a result of the high bearing ratios of the Saharan subgrade soils, the total pavement thickness seldom exceeds 20 cm, and a subbase is not required. Another important result for low-cost roads is that any type of soil can be used for the subgrade. In order to decrease construction costs, both the embankment materials and the subgrade materials can be borrowed along the alignment and in the close vicinity of the road, without any previous selection. In this way, the pavement can be easily constructed on a shallow embankment without previous blading of topsoil, and cuts can be avoided.

BASE MATERIALS

Taking Advantage of Natural Materials

Many authors have stressed the importance of taking advantage of local materials. Tobin has stated that "the biggest financial savings in road construction can probably be made in the selection of pavement materials" (*10*). Humarau said that "the art of the engineer consists for a good part in discovering technologies that will make possible the use of the materials that he finds in the vicinity of the road works" (*11*). However, force of habit, inadequate specifications, and some negative experiences have opposed the use of local materials.

Experience in the previously French, but now Algerian, Sahara indicates that well-selected natural materials can perform as well as, and in some cases better than, stabilized materials such as sand-bitumen and sand-cement. Of course, the natural materials of the Sahara seldom meet the usually accepted base course specifications. But is it surprising that specifications developed under the conditions of humid countries would not work in the environment of the Sahara?

The natural road-building materials that are available in the northwestern Sahara will be reviewed and recommendations that were developed for these materials will be provided in the following sections.

Natural Gravels

Natural gravels in the northwestern Sahara are found as alluvial fans and slope debris. The former are usually rounded and the latter angular. Both usually have a certain amount of eolian sand and an over-sanded, gap-graded granularity. Well-graded natural gravels are exceptional. Natural gravels are usually hard and have a wide range of plasticity indexes.

Calcrete

Calcrete has been studied from the geotechnical point of view by Netterberg in Southern Africa and Horta in North Africa; it is known as caliche in America (*12, 13*). It is formed by the precipitation of calcium carbonate in soils under semi-arid climates.

Calcrete gravels are usually gap-graded and cannot be properly characterized by the Atterberg limits. They exhibit a very wide range of hardness values and their hardness is a function of the grain size. They have the property of self-cementation by dissolution and recrystallization of calcareous fines (*14*). Calcrete materials selection and specification should be based on the carbonates content determination and hardness tests such as the Los Angeles abrasion test (*13*).

Silcrete

Silcrete results from precipitation of silica in soils under particular climatic conditions. It may develop as concretions (Figure 6) and crusts. The former and the latter supply hard gravels upon dismantling that can be specified in the same conditions as natural gravels without any particular problem.

Gypcrete

Gypsum precipitation above shallow water tables results in soft, massive crusts known as gypcrete. Young gypcrete in contact with the water table is not suitable as a road-building material in the northwestern Sahara, but old gypcretes formed during past quaternary times have partially lost their hydration water in contact with the dry atmosphere and are in fact natural plaster mortars.

Old gypcretes may be borrowed as sands (gypsum sand or SY) or silts (gypsum silt or MY) and are composed of eolian quartz sand and gypsum fines. The plasticity index is not significant for proper selection and specification and the gypsum content should be determined. After compaction they provide very stiff pavements with deflections as low as 0.5 mm under a 130-kN wheel axle (*15*). In some regions calcrete and

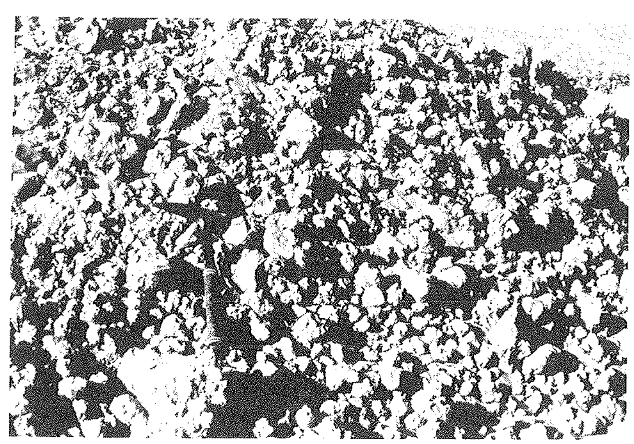

FIGURE 6 Concretionary silcrete gravel on National Road 1, about 75 km to the south of El Golea.

gypcrete are associated or superposed to form gypcalcrete. This type of material is as soft as gypcrete and should be specified by means of the previously mentioned chemical tests (*16*).

Gypsum is soluble and gypsum sand pavements cannot withstand soaking and flooding. The gypcrete pavement of National Road 48 in the Souf region of Algeria was flooded and collapsed once in 1968 to 1969. However, such floods are rare.

Highly Plastic Clays

Clays light in plasticity are not suitable as pavement materials because they are relatively pervious and may soak in a short time after showers. However, highly plastic clays have been used as base courses of bitumen-paved roads and runways (Figures 7 and 8). The calcareous, lateritic clay in the red clay borrow pit shown in Figure 7 was used in the base course of pavements of the airport of Reggane, including runways.

After excavation, the clay appears like a gravel composed of clay mottles of different sizes. This gravel should be quickly mixed with water and compacted with light rollers to avoid crushing. Time is insufficient to soak the mottles deeply after wetting and before the water evaporates. The soaked superficial shell binds the material.

Laboratory CBR tests indicate that the clay specimens that compacted immediately after wetting exhibit higher bearing ratios than the samples kept in plastic bags for 48 hrs after wetting and before molding in spite of lower dry densities (Figure 9). The optimum water content is lower for the former.

Clay bases require good drainage of run-off; for this purpose, the camber of the surfacing should be increased to 3 to 4 percent. This type of material, of course, performs less well than others.

FIGURE 7 Red clay borrow pit in Reggane.

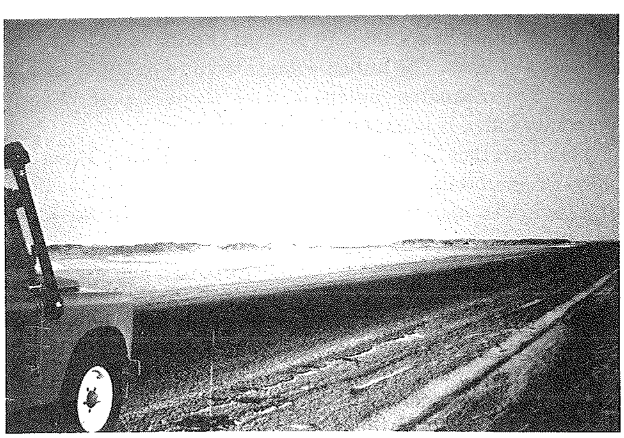

FIGURE 8 Red clay base course and borrow pit on National Road 6 in the vicinity of Sbaa and Adrar.

Classification and Selection of Base Materials

The classification and selection of natural base materials in the northwestern Sahara are based on the following laboratory tests: sieve analysis, Los Angeles abrasion test, carbonates content, soluble sulphates content, and Atterberg limits, W_P and W_L. The last three tests are run on the minus No. 40 fraction. The Los Angeles test should always be run on a specimen of grading A (*17*). Results on other gradings may not be comparable because the hardness of natural materials may be a function of the grain size.

The first selection criterion that should be taken into account is granularity, but this criterion is not restrictive. Materials of any granularity may be accepted as base course materials, provided that certain other conditions are met. The plot of the sieve analysis should be compared with the grading limits of Figure 10. These grading limits delineate three different areas that correspond to three geotechnical families. The grading limits given in Figure 10 are known as the Beni-Abbes grading limits and were introduced by a workshop on the Saharan roads held at that town (*18*).

Family II groups materials whose plots fall within the limits. Families I and III group materials whose plots fall below the lower limit and above the higher limit, respectively. If the plot intersects the lower limit, the material will be considered to belong to Family I. If it intersects the higher limit or both limits, the material will be considered to belong to Family III.

Family I Materials

Materials that belong to Family I do not require further testing. These materials always exhibit satisfactory hardness. Materials of this family are either debris and alluvial gravels with hard cobbles and boulders or calcrete with hardpan.

Gravels with boulders will require previous crushing or screening. Gravels with flat boulders and Los Angeles abrasion losses in excess of 25 percent, such as hardpan calcrete, can be crushed at a low cost by grid rolling. Some gravels without boulders will exhibit hollow granularities because sand fractions are lacking, and will intersect the lower grading limit. If the lack of sand is not compensated by an excess of fines, compaction and priming will be problematic. Gravels without boulders and with a lack of fines should be rejected if they show a sand hunch and are gap-graded. If they are well-graded, they are comparable to hollow-graded materials.

Family II Materials

Materials that belong to Family II are usually gap-graded because of an excess of sand. If the fines have a binding action,

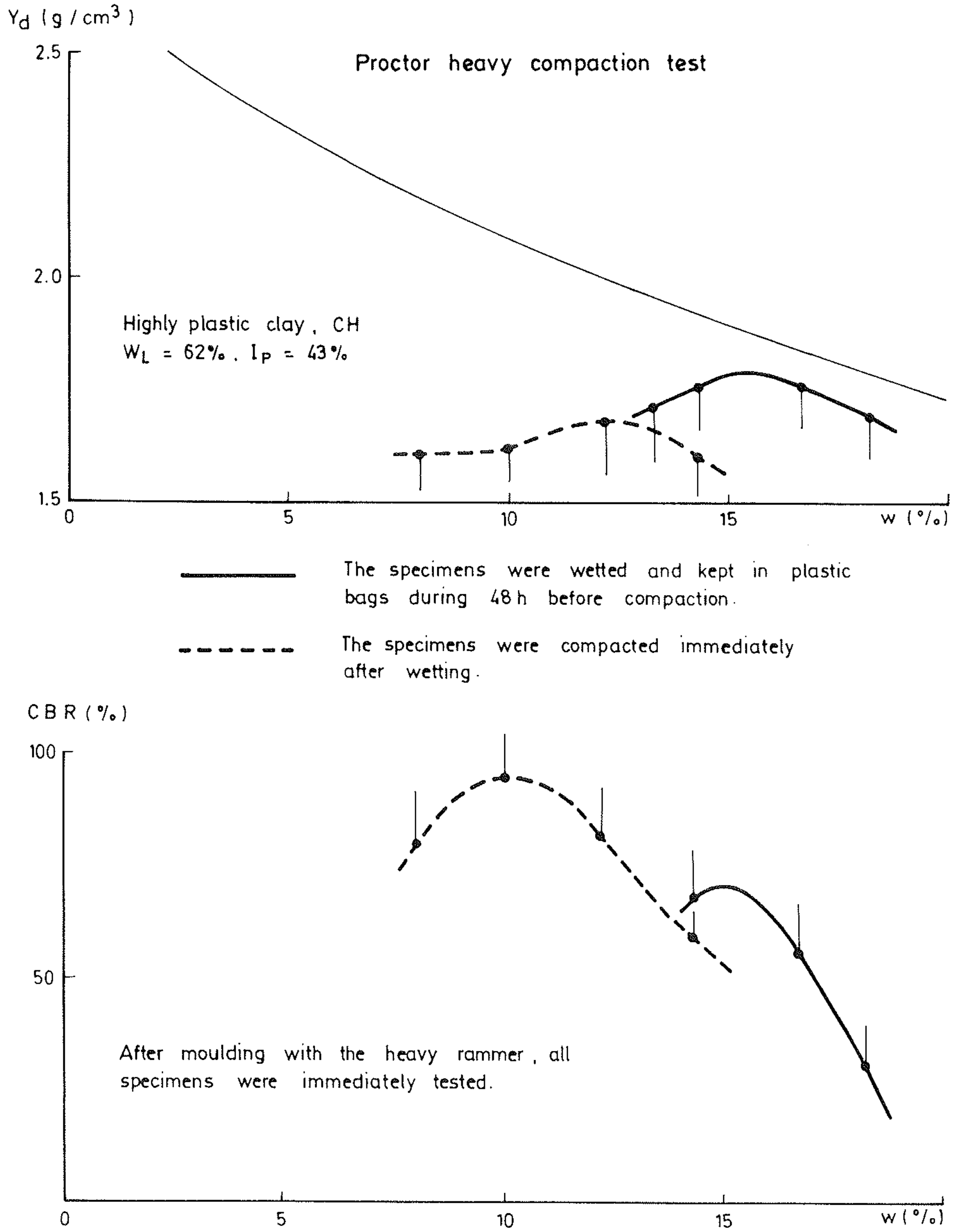

FIGURE 9 CBR of a highly plastic clay for different conditions of wetting.

the excess sand is not harmful. In the case of cohesionless fines, over-sanded and gap-graded gravels are unsuitable as base materials because they loosen after compaction upon drying.

Natural well-graded gravels are exceptional in the northwestern Sahara, but well-graded crushed stone can be produced with adequate equipment. Well-graded materials should have Los Angeles abrasion losses lower than 40 percent. Well-graded natural materials usually comply with this specification and the rocks and boulders subject to crushing should be selected on the basis of this requirement.

Over-sanded, gap-graded materials should also be tested for hardness. If the Los Angeles abrasion loss is 40 percent or more, they should be tested for the carbonates content and rejected if it is not in excess of 70 percent. This is the same criterion that applies to Family III materials. Its application to soft, evolutive gravels is equivalent to considering these as belonging to Family III.

Hard, gap-graded, over-sanded materials with Los Angeles abrasion losses lower than 40 percent should be tested for the cohesion of fines. The fines are considered to be cohesive when either the carbonates content is in excess of 20 percent or the plasticity index is in excess of 6 percent.

Materials that belong to the geotechnical Family II perform best. If materials that belong to this family are available together with materials that belong to Family III, the former should be preferred.

Family III Materials

Family III mainly groups sands and fine soils. Of the soils that belong to this geotechnical family, the following four types can be used as base materials:

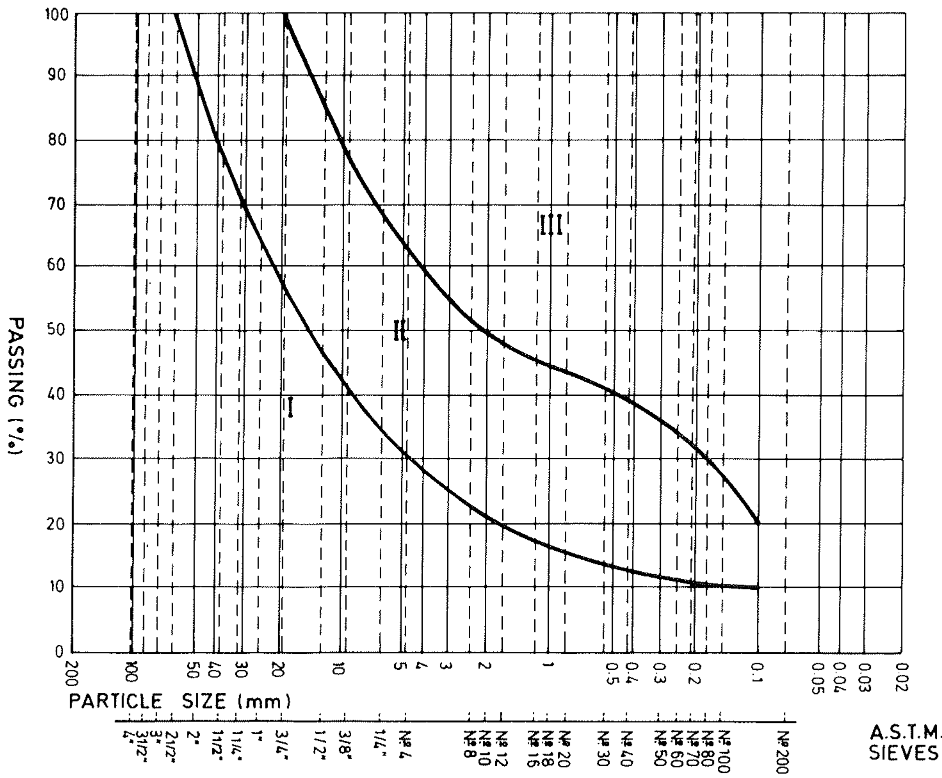

FIGURE 10 Grading limits for base course materials in the northwestern Sahara (Beni Abbes, 1965).

- Calcrete gravels and sands and calcareous fine soils (clays and silts) in which a carbonates content in excess of 70 percent is required;
- Gypsum sands and silts in which a gypsum content in excess of 70 percent is required;
- Gypcalcrete, in which case a total content of carbonates and soluble sulphates in excess of 70 percent is required; and
- High-plasticity clays (CH) that exhibit liquid limits in excess of 50 percent.

Checking the Strength

Some countries have specifications for the bearing ratio of base materials. The CBR should be in excess of 80 or 100 percent, depending on the traffic. The bearing ratios of materials that belong to Family II are usually not problematic. The strength of materials that belong to Family III may be checked by this test. However, the CBR test is not adapted to highly cohesive, stiff materials. In the Sahara, the strength of materials that belong to Family III is usually tested by means of the unconfined compression test on specimens 50 mm in diameter. The unconfined compression strength of materials of Family III should be in excess of 2.5 MN/m^2 after oven drying at 55 to 60° C for 48 hrs and in excess of 2.0 MN/m^2 after drying in the ambient atmosphere for 48 hrs. This recommendation should be considered as a check after selection by other tests as was recommended earlier.

The criteria for the selection of base course materials are summarized in Table 1 and their application is illustrated by the flowchart of Figure 11. The sieve analysis results should be plotted on the graph of Figure 10. Materials that belong to Family I can be accepted without further testing. If well-graded, materials that belong to Family II should exhibit a suitable hardness (Los Angeles abrasion loss <40 percent). As was previously stated, well-graded gravels are very uncommon in the Sahara. Family II materials generally exhibit a more or less accentuated sand hunch; in order to be accepted as base course materials, they should either be hard enough (Los Angeles abrasion loss <40 percent) or contain highly carbonated fines (carbonate equivalent >70 percent). The second condition eliminates sandstone gravels, not soft calcrete gravels. Hard, gap-graded gravels should be tested further for the binding action of fines. Materials that have a calcium carbonate equivalent of 20 percent or less and a plasticity index of 6 percent or less should be rejected.

Materials that belong to Family III should be tested for calcium carbonate and gypsum. Highly cohesive materials with more than a 70 percent calcium carbonate plus gypsum equivalent should be accepted. Materials that exhibit lower values of carbonate plus soluble sulphates content should only be accepted if their liquid limit is higher than 50 percent (highly plastic clays).

It should be emphasized that no upper limit is set to the plasticity index of base materials. Alluvial gravels with plasticity indices in excess of 18 percent have been successfully employed, but sometimes nonplastic gravels could not be used because of an excess of sand.

THE WEARING COURSE

All types of bituminous surfacings have been successfully used in the Sahara, including hot-mix, cold-mix, sand asphalt, and surface dressing. Wearing course thicknesses seldom exceeded 5 cm.

TABLE 1 CRITERIA FOR THE SELECTION OF BASE MATERIALS IN THE NORTHWESTERN SAHARA

Granularity		Los Angeles abrasion (%), grading A	Carbonates content, $CaCO_3$ (%)	Soluble sulphates content, $CaSO_4 \cdot 2H_2O$ (%)	Atterberg limits		Unconfined compression strength σ_R (MN/m^2)
					W_L (%)	I_P (%)	
Family I		n.a.	n.a.	n.a.	n.a.	n.a.	n.a.
Family II	Well-graded	< 40	n.a.	n.a.	n.a.	n.a.	n.a.
	Over-Sanded Gap-graded	< 40	> 20	n.a.	n.a.	n.a.	n.a.
			⩽ 20	n.a.	n.a.	> 6	n.a.
		⩾ 40	> 70	n.a.	n.a.	n.a.	n.a.
Family III		n.a.	> 70	n.a.	n.a.	n.a.	>2.5 after 48h in the oven at 55–60°C
			n.a.	> 70	n.a.	n.a.	
			$CaCO_3 + CaSO_4 \cdot 2H_2O > 70$		n.a.	n.a.	> 2.0 after drying in the ambient atmosphere during 48 h
			n.a.	n.a.	> 50	n.a.	

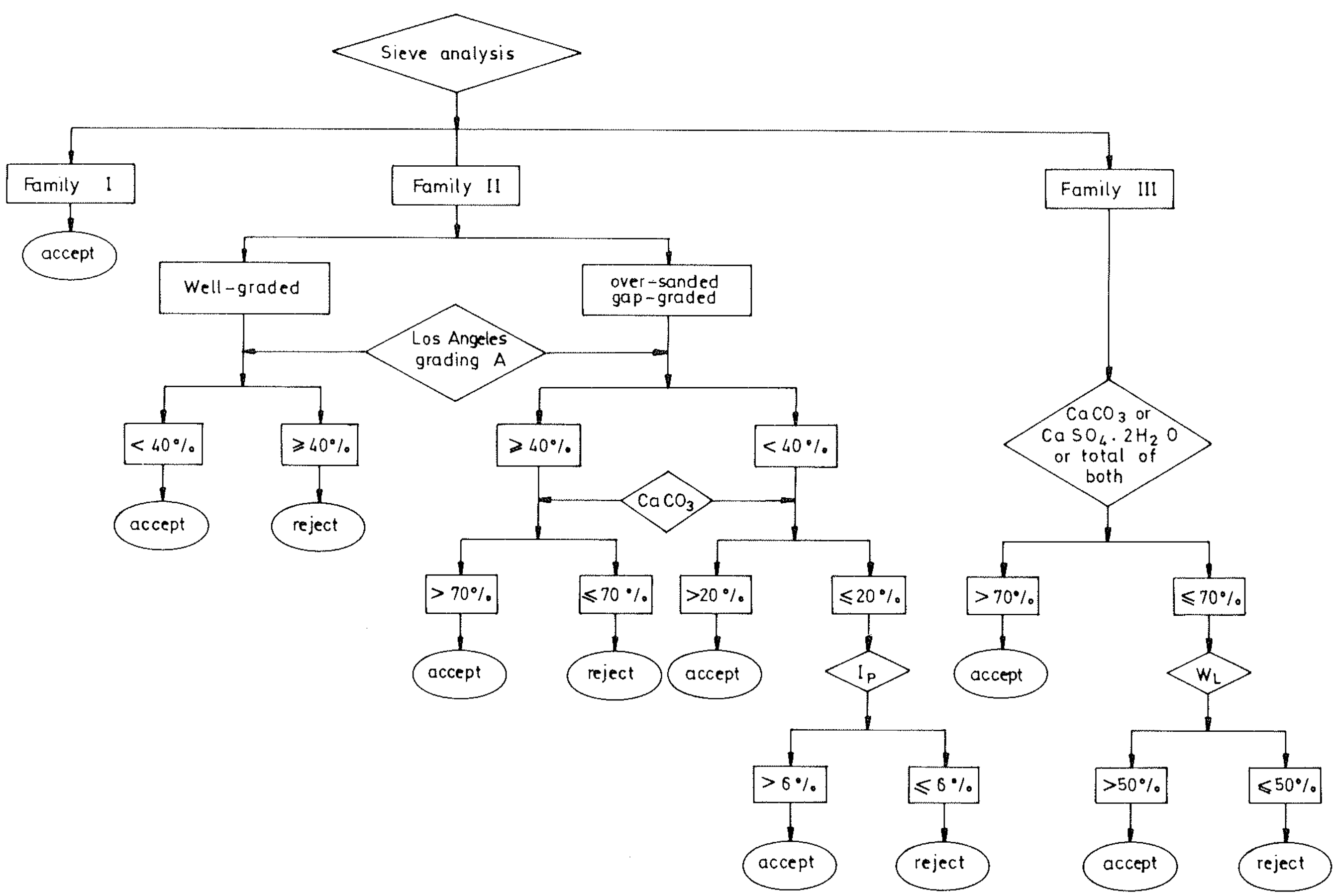

FIGURE 11 Flowchart for the selection of base course materials in the northwestern Sahara.

As a provision for the absorption of sand and dust carried by the wind, it is recommended that the bitumen content be increased. As a result, the surfacing will have the advantage of being flexible. Another advantage is the lower aging rate. Bleeding is not an important problem in the Sahara, because sand is available everywhere. Cold-mix wearing courses do not require seal coats in the Sahara. Their surface will be swiftly sealed by eolian sand.

An adequate binder for surface dressing is penetration bitumen. The viscosities of cut-back bitumens may be too low under the high temperatures of the Sahara and should only be used in the cool season. Emulsions risk premature setting because the chips are usually covered with dust and mixed with eolian sand. The successive sprayings of binder and spreadings of chips should be done without interruption to avoid pollution by sand.

The wearing courses of the desert roads are exposed to heavy erosion by sand-loaded wind. The differential erosion of softer bitumen exposes the harder chips to stripping by the traffic. Deflation over bituminous pavements may cause higher rates of wearing than traffic alone.

PAVEMENT STRUCTURES AND PERFORMANCE

The pavement structures in the northwestern Sahara are usually very light. Depending on the traffic, total pavement thicknesses of 15 to 25 cm are required by current design methods (*19*). The pavement full depth is generally constructed with materials that comply with the criteria discussed earlier. The embankment is compacted at natural water content, except the top layer. This layer usually has a high strength and, as previously discussed, a subbase is not required.

The road network of the Algerian Sahara, including more than 7000 km of bitumen paved roads is shown in Figure 12. Distance and some other problems prevented consistent monitoring of the Saharan road pavements. However, the prevailing opinion on the performance of the Saharan road pavements in Algeria is very positive (*20*). Maintenance has been restricted to sand drift control and resealing. In spite of traffic growth, pavement strengthening has not been contemplated.

Recent traffic figures of the Saharan roads in Algeria and the dates of construction, pavement materials, and visually surveyed condition are given in Table 2. The roads and count stations referred to in this table are shown in Figure 12.

The traffic on the Saharan roads is generally heavy to very heavy. About 80 percent of the traffic count stations display heavy traffic levels in excess of 30 percent; this rate exceeds 50 percent in a third of the count stations. It should be emphasized that the legal axle load in Algeria is 130 kN. Calcrete and natural gravels perform best as pavement materials and are able to carry high volumes of traffic.

In spite of hygroscopical cracks, the behavior of gypcrete pavements is good, provided that the subgrade water content remains low (*8, 15*). This is not always the case for Road N3.

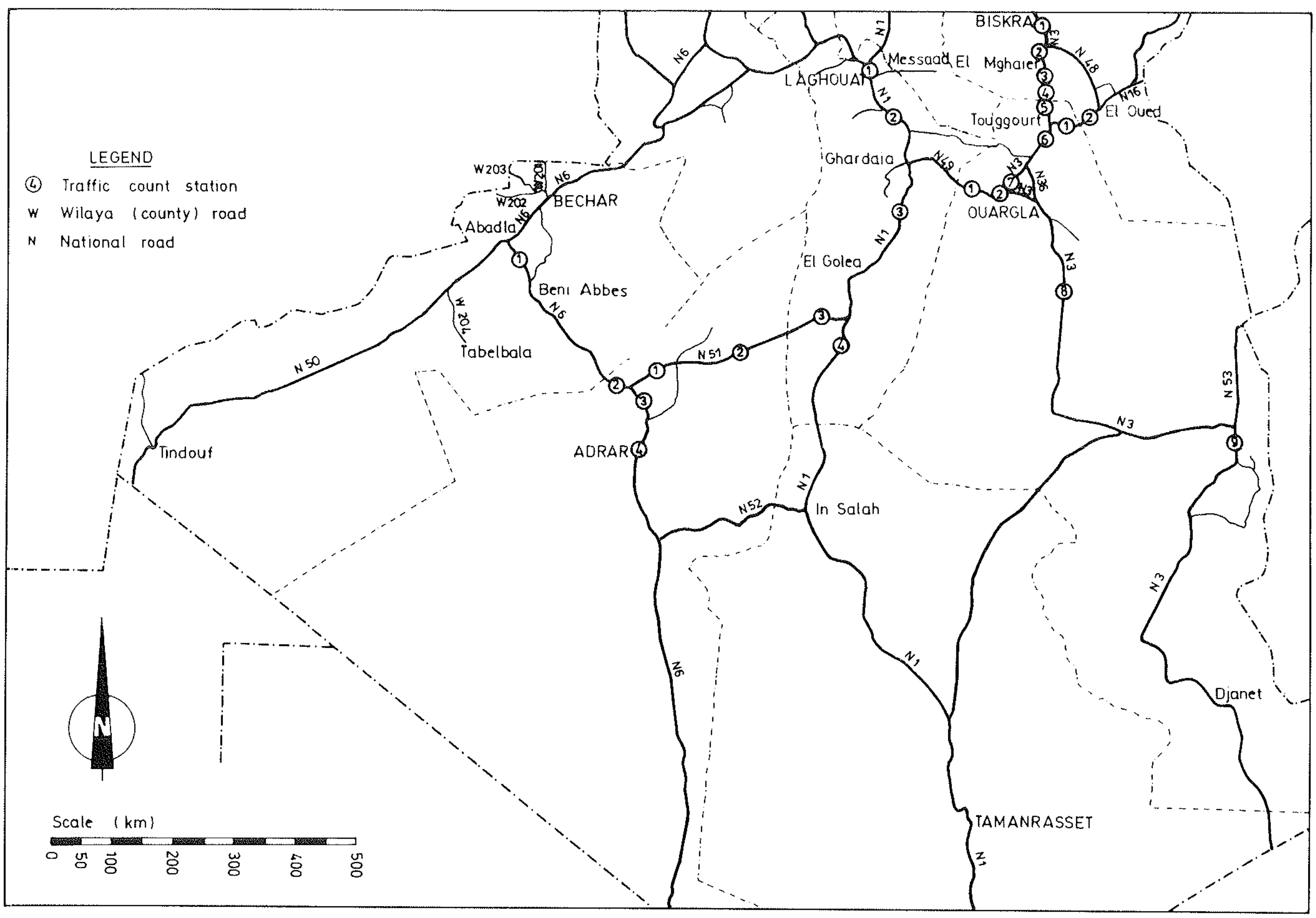

FIGURE 12 Algeria's Saharan roads.

TABLE 2 TRAFFIC FIGURES (1985) AND PAVEMENT PERFORMANCE IN THE ALGERIAN SAHARA

Road	Wilaya	Count station	Average daily traffic	Heavy traffic (%)	Date of construction	Pavement materials: Wearing course	Pavement materials: Base course	Pavement condition	Date of last survey
N1	Laghouat	1	1106	32	1960-62	hot mix	II, III (calcrete)	local shear: water entrappment	1979
		2	1724	45	1958-59	idem	II	good	1979
		3	360	69	1956-59	surf. dress.	binder, II	good	1979
		4	-	-	1974	cold mix	non standard	failed	1982
N3	Biskra	1	2844	-	1957	surf. dress.	II	good	1979
		2	3187	31	1957	hot, cold mix, sand asph.	III (gypcrete)	fair	1979
		3	2720	29	1957	idem	idem	fair	1979
		4	1388	45	1957	idem	idem	fair	1979
	Ouargla	5	1192	28	1958-59	idem	idem	fair	1979
		6	863	62	1958-59	idem	III (gypcalcrete)	good	1976
		7	1798	38	1958-59	idem	idem	good	1979
		8	729	40	1958-59	sand asphalt	chem. stab. soil	failed	1969
		9	128	64	1962-63	surf. dress.	II	good	1969
							III (clay)	good	1969
N6	Bechar	1	258	83	1960-62	surf. dress.	II	good	1969
		2	184	65	1963-64	idem	II	good	1969
		3	221	62	1965-66	idem	II	good	1969
		4	663	26	1966	idem	III (clay)	good	1969
N16	Ouargla	1	1124	30	1963-64	idem	III (gypcrete)	good	1976
	Biskra	2	1000	24	1963-64	idem	idem	good	1976
N36	Ouargla		747	33	1958-59	idem	III (gypcalcrete)	good	1969
N48	Biskra		756	67	1957	hot mix	III (gypcrete)	fair, flooded 1968-69	1976
N49	Laghouat	1	715	48	1958-59	cold mix	pen. macadam	good	1976
							II	good	1976
	Ouargla	2	821	58	1958-59	idem	III (gypcalcrete)	good	1976
N50	Bechar		327	95	1965-67	idem	I, II	good	1969
N51	Adrar	1	161	58	1967-68	idem	I, II	good	1982
		2	92	64	1968-69	idem	I, II	good	1982
						idem	soil-bitumen	failed : salt damage	1982
	Laghouat	3	115	20	1968-69	idem	I, II	good	1982
						idem	non standard	rutting	1982
N52	Adrar		41	46	1980 (?)				-
N53	Ouargla		74	40	1963-64	surf. dress.	II	good	1969
W201	Bechar		984	55	1975 (?)		II		-
W202			1040	42	1975 (?)		II		-
W203			610	21	1975 (?)		II		-
W204			40	0	1975 (?)		II		-

The pavement of this road developed settlements and alligator cracks along some oasis stretches with shallow water tables. Clay base courses may be able to carry low volumes of traffic for a few years.

DRAINAGE AND SHALLOW WATER TABLES

In wide, small valleys, the drainage of run-off at the pavement level is usually preferred to save the cost of culverts. In this case, the shoulders should be surfaced for protection against scour. The pavement of these sections should be constructed with materials of a low susceptibility to water. Natural gravels with a low clay content and calcrete can be used. The pavement should be designed on the basis of soaked CBR. However, these sections are exceptional and the general situation is that of low embankments and deep water tables very much in excess of 7.5 m, without any influence on the subgrade water content.

In some places, such as oases and salt dry lakes (sabkhas), the water table is shallow. In order to provide efficient protection against soaking and salt migration, the pavement should be placed on embankments of sufficient height and constructed with clean, coarse soils such as eolian sand.

Some of the base materials used in the Sahara, for instance gypcrete, are very susceptible to moisture. For fast draining of run-off, the surface camber should be about 3 percent and the surfacing impervious. The latter requirement is usually met because bitumen proportioning is increased to account for sand absorption.

DAMAGE BY SOLUBLE SALTS

The easiest test to detect soluble salts is tasting. Tasting is a very sensitive way to detect the amount of salt that would be harmful to the pavement, such as a 0.5 to 1.0 percent sodium chloride equivalent. Soluble salts are able to migrate through menisci of soils provided the liquid films are continuous. In this way they get at the pavement. Upon evaporation of the pavement moisture, salt crystals grow at the surface or between the base and wearing courses, and cause heaving of the latter and subsequent damage to the pavement (*21*).

Sometimes the salts come either from the compaction water or from the base materials. In these conditions they can be easily eliminated by sweeping after compaction and drying. Difficult situations arise where the salts originate in shallow water tables. Impervious membranes or very pervious cut-off layers are necessary to stop salt migration (*20*).

COMPACTION AT NATURAL WATER CONTENT

Water is a scarce commodity in the desert. It not uncommonly has to be pumped from deep aquifers in drill holes several tens of meters long and transported over distances of several tens of kilometers. In addition, evaporation of compaction water may be as high as 50 percent in hot, windy weather. This is why dry compaction of embankments has been implemented in the Sahara for a long time. Actually, dry compaction means compaction at natural water content, usually in the range of 0.5 to 2.0 percent, without additional wetting.

Experience has shown that dry compaction is feasible with heavy compactors. Recent studies by the Laboratoire des Ponts et Chaussées of France confirmed this observation (*22*). Excellent results can be achieved with heavy dynamic compactors and nonplastic coarse soils, namely eolian sand. Dynamic compaction is effective to a depth of 40 cm but the upper layer of about 10 cm remains loose. This layer should be compacted through the next layer or by static rollers after wetting at the optimum moisture content. Silty and clayey coarse soils are more difficult to compact at natural water content, but it is still possible to achieve satisfactory results for earthworks.

The compaction of base materials at natural water content would therefore be possible for cohesionless materials that belong to Families I and II and dry bound macadam. However, compaction is excluded for cohesive materials such as calcrete and gypcrete. The strength of these materials is mobilized by water.

Cisse provided some figures for savings that resulted from dry compaction of the Tahoua-Arlit road in the Saharan region of the Republic of Niger (*23*). The water supply may amount to 10 percent of the total construction cost in some stretches. The saving in relation to the total cost of earthworks was 38.8, 38.6, and 14.4 percent for the three different stretches of this road.

CONCLUSIONS

The specific characteristics of the desert environment and the distinctive features of road-building technology in the northwestern Sahara have been discussed. The feasibility of taking advantage of the characteristics of the desert environment to decrease road-building costs has been shown. Long distances and scarce economic activity do not contribute to low construction costs. However, the stable terrain relief with few obstacles and the scarce number of towns to serve result in a large degree of freedom in the location of roads and to build roads of shorter length. These conditions also permit roads to be located closer to road-building materials deposits, which decreases hauling costs.

The aridity of the desert should also be considered as a remarkable advantage to road engineers. Bridges can be replaced by fords. The drainage of pavements is simplified as a result of the absence of springs and the low frequency of shallow water tables. A further advantage that results from aridity is the usually dry state of the subgrade soils, which exhibit high strength. Strong soils require light pavements even when the traffic level is relatively high.

The very low water contents that result from high evaporation rates enable the use of a wide range of natural materials in the base course. Some of these materials contain important amounts of relatively soluble minerals, such as gypsum, that would dissolve under the humid climates of Mediterranean northwest Africa, with subsequent collapsing and formation of hollows and caves. Most of the base course materials used in the Sahara contain significant amounts of clay and practically all of them have or should have a plasticity index in excess of 6 percent.

However, the desert environment also has some disadvantages and presents some difficulties that were mentioned earlier. If these disadvantages are not considered, it is not possible to speak of low-cost roads in the Sahara, and maintenance costs will increase to prohibitive levels.

Eolian sand drift is a very difficult problem that should be considered at the design stage and carefully studied and experimented with by trial and error in sections that are exposed to drifting.

As a result of aridity, the water supply is very expensive, which is why dry compaction has been experimented with and studied. Compaction of the earthworks and a few base materials at natural water content is feasible.

Inattention to soluble salts, which may be contained in materials and compaction water, would also result in higher maintenance costs because these salts may cause damage to bituminous surfacings.

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Rural Roads in Cement Concrete: A Technique That Can Be Adapted to Developing Regions

F. FUCHS AND P. SION

Interest in concrete roads exists for many reasons, especially in countries with adequate cement supplies. Concrete roads offer several advantages to other solutions from both technical and economic points of view. In Belgium, 50 percent of the rural road network of land reallocation regions consists of cement concrete. The concrete slabs are usually laid directly on the soil without the provision of a road base. The initial construction cost of these cement concrete roads is very competitive with that of other types of structures that comprise a bituminous pavement. Maintenance costs are very low or nonexistent, and the service life is estimated at more than 40 years. The construction of this large network of rural roads since 1960 has enabled the development of design and construction methods and specifications that are particularly well-adapted to this type of road. These methods can be transferred not only to other industrialized countries, but also to developing regions. The thickness of the concrete pavement varies from 16 to 20 cm, depending on the traffic volume and the modulus of the subgrade soil. The usual practice is to improve the subgrade only when the modulus of the soil is lower than 20 MPa (California bearing ratio value < 2). Slab length is limited to 5 m and the joints are made by inserting plastic strips into the fresh concrete. The concrete can be laid in several ways, ranging from labor-intensive methods to techniques that require specific equipment. In this respect, the use of slip-form pavers currently makes it possible to achieve daily productions of about 400 to 600 m with a reduced concreting team. Finally, an analysis is provided of the elements in the selection of concrete pavement technology for minor roads. The analysis is based on technical and economic criteria connected with each particular context, and on the availability of materials and labor.

Over the past few years, interest in the construction of cement concrete roads has revived in various countries for use in motorways and major roads, and for low-volume rural roads (Figure 1). This trend is especially apparent in countries with adequate cement supplies and in which the import of oil products is an additional drain on foreign exchange resources (*1, 2*).

Belgium has many years of experience in the construction of concrete roads, dating back virtually to the beginning of the century. Therefore, it is not surprising that the engineers entrusted with the design of the first rural land reallocation projects in the 1950s resorted to this technique for the construction of part of the rural road network (*3, 4*). This technique offers several specific advantages to other solutions, from both technical and economic points of view:

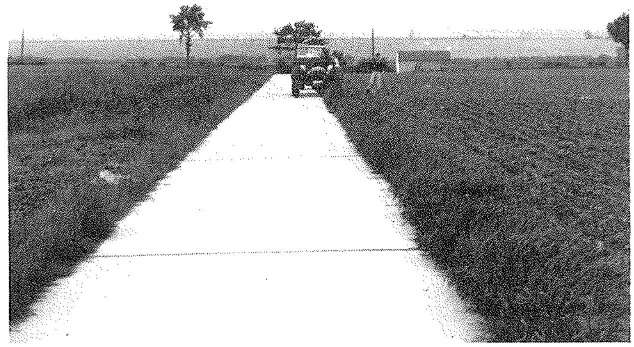

FIGURE 1 Cement concrete rural road.

- A good load distribution, which eliminates the need for thick and expensive bases;
- A great resistance to deformation and wear at any temperature;
- The degree of solidity is the same at the edges of the pavement as it is at its center; and
- An insensitivity to stagnant oil, clay, or fecal matter.

Concrete pavements also meet the following economic criteria:

- An estimated service life of more than 40 yrs,
- Virtually nonexistent operating and maintenance costs, and total construction costs that are generally lower than those of flexible pavements, and
- A competitive initial investment cost, as a result of an advanced laying technology.

The trend toward the use of concrete has become even more marked over the past 10 years as a result of the relative stability of the construction costs of concrete rural roads in regard to the consumer price index and the price index of the principal construction materials (Figure 2). In terms of initial investment costs, this stability has given concrete roads a clear advantage over flexible, bituminous pavement, as shown in Figure 3. In July 1985, the construction costs under Belgian conditions of structures a, b, and c in Figure 3 were in the ratio of 2.1: 1.5: 1; the unit costs per running meter of 3-m-wide rural roads were 2,650 BF/m, 1,887 BF/m, and 1,268 BF/m, respectively (1 \$ US ≈ 40 BF). The most recent methods for the design and construction of rural concrete roads are presented. The criteria for evaluating the adaptability of this technique to other

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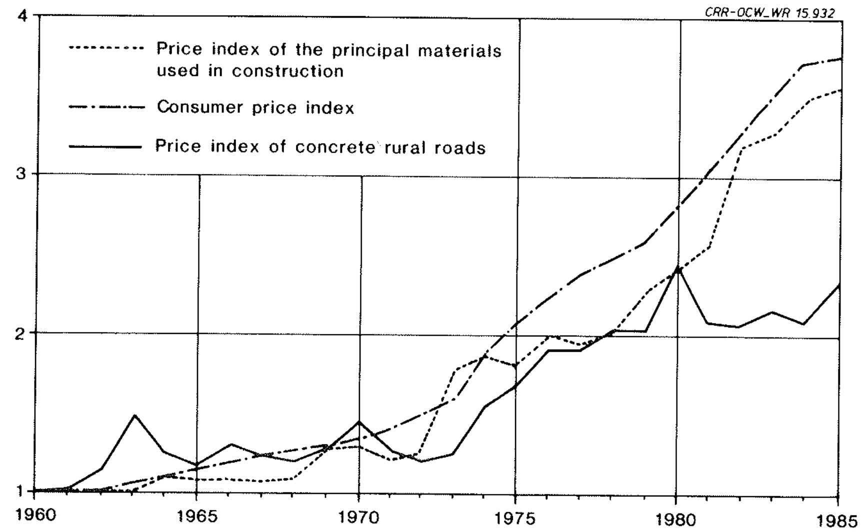

FIGURE 2 Evolution of the construction cost of concrete rural roads from 1960 to 1985.

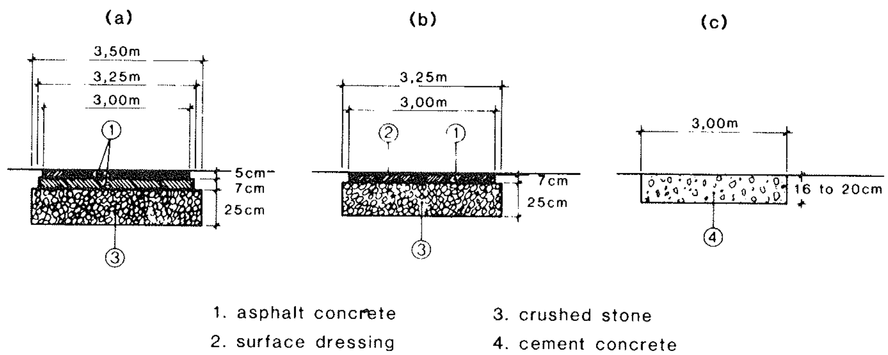

FIGURE 3 Typical structures of asphalt or cement concrete rural roads.

countries, particularly developing countries, are then analyzed on the basis of the technical environment and economic context of each particular case.

DEVELOPMENT OF THE CONSTRUCTION OF CONCRETE RURAL ROADS

Cement concrete pavements have comprised a major portion (50 percent) of the rural land reallocation projects in Belgium since 1958. The initial choice was based on the good performance of this type of pavement on state and secondary roads in the 1930s. The development of cement concrete and bituminous pavement construction from 1975 to 1985 is depicted in the histogram in Figure 4.

Although the percentage of concrete pavements remained stable at 46 to 54 percent between 1958 and 1975, it has increased since 1976, when the first effects of the oil crisis were felt. This percentage increased to an average of 57 percent between 1976 and 1980, and reached 79 percent during the next 5-yr period, with a maximum of 83 percent in 1982.

The histogram also shows a very marked reduction in road construction in rural land reallocation areas, from 400 km in 1977 to 120 km in 1984. This reduction is a result of drastic cuts in funds allocated by regional authorities for rural land reallocation projects.

THE DESIGN OF CONCRETE RURAL ROADS

Although a great many design methods exist, their application to rural roads leads to problems that mainly stem from the limited resources available to the designers and the relatively high study costs. It is also hazardous to extrapolate fatigue laws to low-stressed pavements (10^3 to 10^4 heavy vehicles in 40 years). Finally, most methods require considerable means to evaluate soil-bearing capacity and traffic.

A guide to the design of rural roads has been developed in Belgium to provide a functional classification of rural roads (*5*). For example, these roads may function as service roads, farm or forestry roads, roads intended for housing estates or tourism, or roads that are or are not used by public transport vehicles or heavy traffic. Roads were grouped into the following classifications:

- Primary rural roads serve villages and business centers, and link them to each other and to the state and provincial networks. The width of these roads is never below 5 m and they consist of two lanes.
- Secondary rural roads serve hamlets and housing estates, and link them to each other, to the villages, and sometimes to the state and provincial networks. Secondary roads consist of one or two lanes and their width may be 3 or 5 m.
- Tertiary rural roads essentially serve land parcels used for

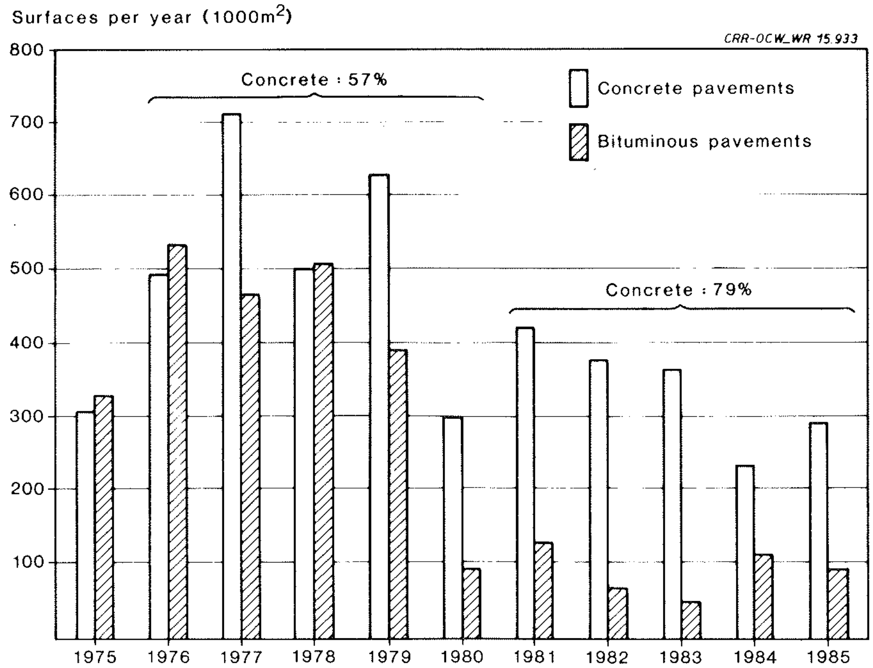

FIGURE 4 Construction of rural roads in land reallocation regions from 1975 to 1985.

farming or lumbering, and link them to the farms and other networks. They consist of one traffic lane and their width is generally 3 m. The daily number of vehicles of all categories traveling in both directions (V_{AC}) on each class of road is given in the following table.

Rural Road	V_{AC}	*Width*
Primary (two traffic lanes)	300-900	≧5 m
Secondary (one or two traffic lanes)	50-300	3-5 m
Tertiary (one traffic lane)	<50	3 m

Agricultural Traffic

Another subject addressed in the guide to the design of rural roads is the study of agricultural traffic. The traffic to be considered for the design calculation can be determined from the following equation:

$$N_R = V_{AC} \times 300 \times a \times c \times T$$

where

N_R = the equivalent number of standard axles;
300 = the number of days of the year, considering the decrease in traffic during weekends and public holidays;
a = the percentage of commercial vehicles (laden weight > 3.5 t), which depends on the type of farming;
c = the cumulative factor, which accounts for the foreseeable annual traffic growth during the service life of the road,

$$c = \frac{(1 + i)^d - 1}{i}$$

where

i = annual rate of traffic growth, and
d = service life estimated at 40 years for concrete pavements; and
T = the loads factor, which characterizes the deterioration mechanism of the road,

$$T = \bar{n} \, \Sigma f_i \left(\frac{P_i}{P} \right)^m$$

where

$\bar{n}$ = the average number of axles per commercial vehicle,
f_i = the proportion of axles with a load P_i, and
P = the standard axle load.

The standard axle considered in Belgium in the design of rigid structures is 13 t. The value of exponent m, which characterizes the damaging effect of axles, varies according to the authors and has been taken as equal to 14 (*6*). When no data are available on the number of vehicles of all categories (in the case of a new road), the evaluation of traffic is based on the type of farming and on the number of loads to be transported. The number of commercial vehicles is evaluated by accounting for the types of vehicles normally used in the region.

Soil Bearing Capacity

The bearing capacity of the subgrade soil is one of the most difficult factors to assess. Many existing methods also require the use of equipment that designers do not possess. Therefore, an original approach to the evaluation of soil bearing capacity has been tried in Belgium since 1976. This approach consists of interpreting soil maps (*7*). Although pedology is a science that aims at the classification of soils for agricultural purposes, it has been observed that a certain relationship exists between the first two pedological indices and soil bearing capacity.

The first index actually defines the morphological characteristics of the soil, such as silt, sand, and clay. The second index, which pedologists call the drainage class, is actually a precious piece of information about the presence or absence of a permanent or temporary water table. The remaining indices are not directly useful in the determination of bearing capacity, although they provide some information. This evaluation is of course only a rough estimate, but it has a great advantage; it is quick and does not require expensive investments. Furthermore, these values make it possible to avoid the most frequent major errors in the design of low-traffic roads.

The triangular diagram that is used by pedologists to determine the first pedological index is shown in Figure 5. The estimated values of the soil modulus as a function of the two pedological indices are given in Table 1. Improved subgrades and stony embankments are characterized by an 80-MPa modulus.

The relationships between pedological indices and bearing capacity were established on the basis of a large number of measurements with the light percussion sounding apparatus of the Belgian Road Research Center (*8*). These measurements enabled the California bearing ratio (CBR) and the soil modulus to be determined on the basis of the following relation: E_s (MPa) = 10 × CBR. Although the constant in this relation is subject to criticism, it appears that it can be applied to Belgian soils without the risk of a major error (*9*).

Characteristics of the Concrete Used for Agricultural Roads

The average composition of concrete used for agricultural roads is as follows:

Aggregates	70 percent
Sand	30 percent (by weight)
Cement (Class 30)	300 kg/m^3
Average compressive strength after 90 days	45 MPa (6,390 psi)
Average tensile strength	6 MPa (852 psi)
Modulus of elasticity	30,000 MPa (4,260,000 psi)

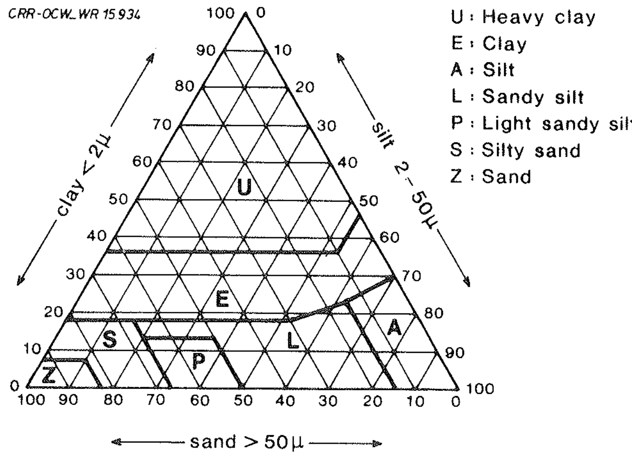

FIGURE 5 Triangular diagram of the grading of soils.

Design Chart

A design chart has been developed (Figure 6). It shows the thickness to be adopted for the pavement depending on the number of equivalent axles of 13 t and the subgrade modulus expressed in MPa. This chart resulted from the application of a model based on layered elastic slab theory. Stresses that result from the effect of temperature gradients have not been accounted for because the length of the slabs has been limited to 5 m. However, a risk factor has been accounted for by assuming a 2.5 percent cracking in the slabs, which can be tolerated by roads in a rural environment.

The subgrade has to be improved in cases in which the value of the modulus is lower than 20 MPa. This can be done by stabilizing the natural soil with lime or cement, or by providing a subbase. The thicknesses found by using the guide are the same as those usually adopted for agricultural roads.

LAYING THE CONCRETE PAVEMENT

Cement concrete can easily be laid in several ways, ranging from the simplest labor-intensive methods (Figure 7) to highly

TABLE 1 EVALUATION OF THE SOIL MODULUS

Soil (Basic symbols cf. figure 5)	Drainage class			
	dry to very dry soil good drainage	moderate and imperfect drainage	wet soil with permanent or temporary water table	very wet to extremely wet soil with permanent water table
Z, S, P, L	40	40	20	≤ 10
A, E	40	20	≤ 10	≤ 10
U	20	≤ 10	≤ 10	≤ 10

Note: 1 MPa = 142 psi.

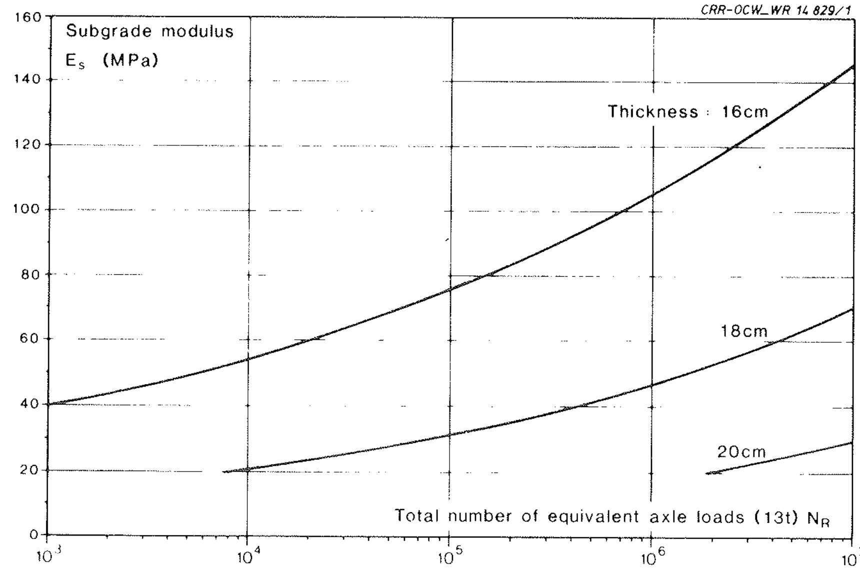

FIGURE 6 Design chart for concrete rural roads.

productive methods that require specific equipment. The choice of one construction method over another depends mainly on the availability and qualifications of labor, the available equipment, the amount of work to be performed, and the desired rate of construction of the planned network. In this respect, although experience has shown that it is possible to construct small concrete roads of a high quality with reduced equipment, it appears that the development of a concrete road network at a reasonable rate inevitably requires the use of specific laying machines.

Until about 1960, concrete rural roads were laid in Belgium with so-called conventional machines that rolled on fixed form work. The concreting train generally consisted of several machines; the first distributed the concrete, the second compacted the concrete by vibration, and the third created the crack inducer of the joints in the fresh concrete (Figure 8).

Lightly modified asphalt finishing machines were used since 1965 to lay concrete pavements (Figure 9). Small slip-form pavers that were developed from machines used on large motorway construction sites were used in the 1970s to lay widths of 3 to 5 m with an average daily production of about 400 m (Figure 10). The various laying methods for concrete rural roads with a width of 3 m are compared in Table 2.

Construction of the Joints

The joints are generally created by inserting a plastic strip in the fresh concrete to a depth of one-third of the thickness of the pavement. Joints are made 5 m apart and are neither dowelled nor sealed. The machine has to stop when a joint is created in the fresh concrete and this reduces productivity. Some contractors saw the joints in the hardened concrete and indicated that the sawing cost is compensated by the increase in daily production.

It is difficult to compare the two joint construction methods because daily production can be influenced by many other factors, such as the production capacity of the concrete mixing plant, the distance between the plant and the construction site, the number of lorries assigned to the transport, the access facilities to the construction site and the length of the roads to be concreted.

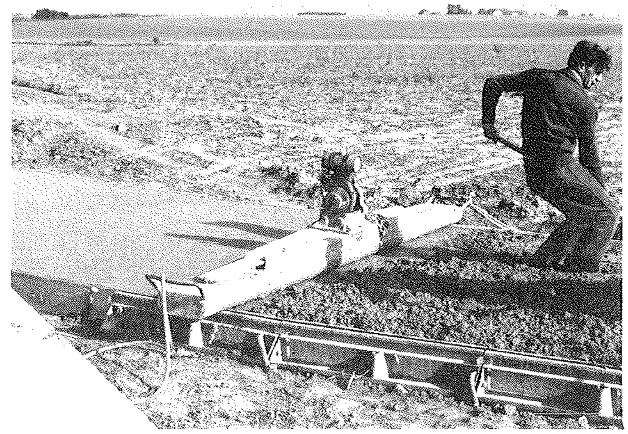

FIGURE 7 Manual application of concrete by means of a vibrating beam.

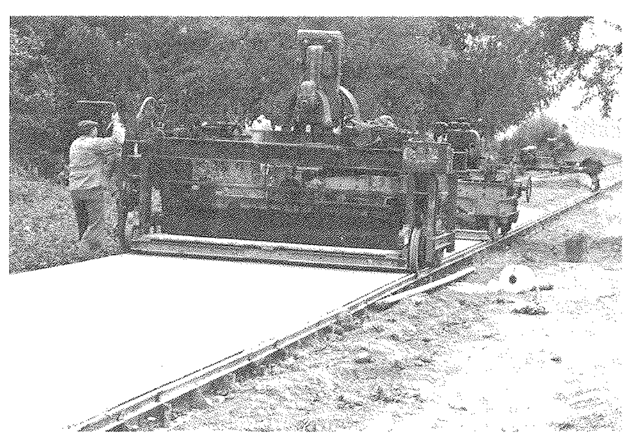

FIGURE 8 Concreting train on fixed form work.

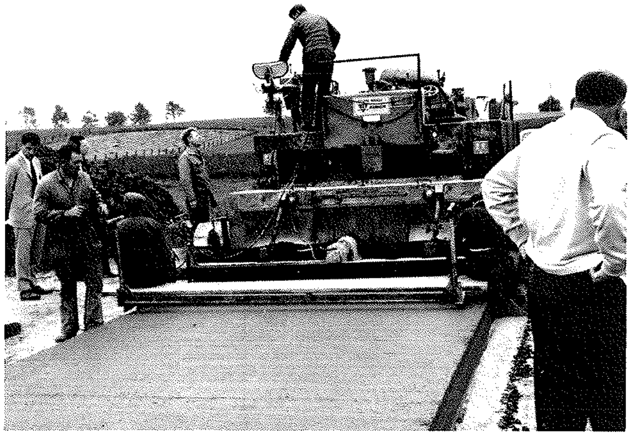

FIGURE 9 Modified asphalt finishing machine for laying concrete pavement.

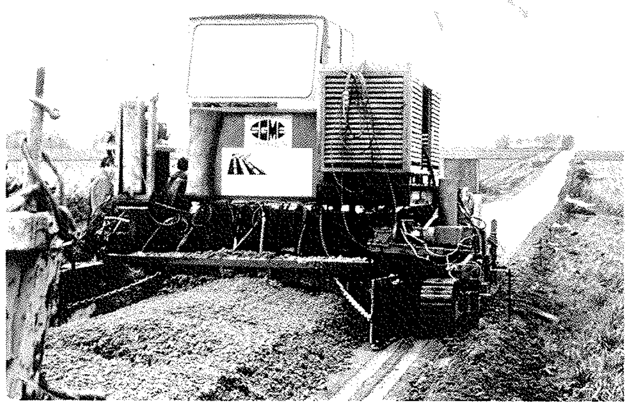

FIGURE 10 Small slip-form paver for roads 3- to 5-m wide.

ELEMENTS IN THE SELECTION OF CONCRETE RURAL ROADS

The selection of a road construction technique should be based on criteria that are associated with both the technical environment and economic context of the country or region concerned. These criteria include possible sources of materials (aggregates and binders) and supply conditions, financial and energy savings, environmental protection and equilibrium in the balance of payments.

The circumstances under which concrete pavements are able to compete with other techniques in the construction of rural roads are examined on the basis of Belgian experience. These circumstances can then be quantitatively assessed in each particular context, on the basis of a technical and economic feasibility study.

Technical Aspect

The construction of concrete roads first requires the availability of adequate cement and water supplies. Apart from that, concrete can tolerate the use of various types of aggregate or sand fairly well, as long as these materials meet certain criteria, such as grading and cleanliness criteria. In regions where hard aggregates are scarce, concrete pavements can be advantageous because they require fewer aggregates and in most cases allow the use of local aggregates that are less hard. In regard to soil conditions, concrete pavements can tolerate a certain local loss of bearing capacity during a thaw or in case of flooding, for example, and are consequently less sensitive to the underlying soil and climatic conditions. It should be noted that it is generally on soils with a low bearing capacity that concrete is competitive (because of its great ability to distribute loads), provided the soil does not lead to differential settlements by its compressibility and heterogeneity. Concrete stands up better to severe climatic conditions than flexible materials, especially in hot climates (no risk of rutting) and in regions where flooding is likely to occur.

In the case of seasonal agricultural traffic, the stiffness of the pavement is an advantage in regions where the heaviest loads travel during the wet season. However, it should be remembered that concrete pavements are very sensitive to underdesign and overloads that are not specifically considered in the design calculations. The addition of a slight thickness guards against possible overloads. In densely populated regions where traffic cannot be diverted, 2 to 3 days must be allowed for the necessary curing process of the concrete (*10*).

Economic Aspect

The decisive criterion for the selection of a construction technique is the total cost of the structure, calculated over a long period. The total cost includes the construction and maintenance costs, and possibly user costs. This calculation should also account for the parts of foreign exchange and local currency, and the impact of the adopted strategy on the economy of the region (use of home materials, balance of payments, etc.).

Apart from this criterion, the selection of the type of pavement can also be influenced by the way construction projects are subsidized. In this regard, the selection of concrete structures is favored by financing structures that subsidize all or a portion of the investment costs for pavements except maintenance, which is often the responsibility of local organizations. However, the use of flexible structures is advantageous in cases in which financial resources are limited or the discount rate is high. This is because a time-staged construction strategy can be applied to the use of flexible pavements. This strategy of construction, maintenance, and strengthening nevertheless requires, even for low-volume roads, that the necessary maintenance funds be available in time to keep the pavement at an acceptable level of service.

However, experience has shown that even when the construction cost of concrete pavements was slightly higher than that of flexible structures, the absence of maintenance has amply confirmed the advantage of a higher initial investment, even if it is 10 percent higher. Moreover, user costs are reduced by the maintenance of a high level of service during the entire life of concrete roads.

CONCLUSIONS

Most rural roads in Belgium are now constructed in cement concrete. The use of high-production laying techniques has led to construction costs that are lower than those of equivalent flexible structures. The performance of rural roads in concrete has also proved to be excellent in the long term, because they require virtually no maintenance.

TABLE 2 COMPARISON OF THE LAYING METHODS OF 3-m-WIDE CONCRETE RURAL ROADS

	Conventional concreting train on fixed formwork	Modified asphalt finishing machine	Slip-form paver
- Formwork	yes	-	-
- Subgrade finish	important under the formwork	important on the track rolling paths	wire-guided
- Extra width necessary	0.50 m	0.75 to 1 m	1.40 to 1.60 m
- Number of machines	1, 2 or 3	1	1
- Concrete composition	no particular restriction	continuous grading	continuous grading
- Slump	0 to 1 cm	2 to 3 cm	2 to 3 cm
- Vibration mode	vibrating beam	vibrating beam	internal vibrators
- Vibrating frequency	50 to 66 Hz	adjustable from 0 to 66 Hz	adjustable from 100 to 200 Hz
- Theoretical maximum working speed	0.60 m/min	1.8 m/min	2.8 m/min
- Theoretical maximum production (8 hours of work)	288 m	864 m	1344 m
- Average practical daily production	125 m	225 m	400 m
- Production capacity of the concrete mixing plant	10 to 15 m^3/h	20 to 30 m^3/h	60 m^3/h
- Concrete transport	0.5 lorry/km	1 lorry/km	2 lorries/km
- Labour (Men x Days)	72 MD/km	30 MD/km	18 MD/km

These construction techniques are also suitable for developing countries that have plentiful cement supplies, but often limited maintenance funds (*11*). Rural roads in concrete can be constructed by local contractors using either very simple labor-intensive equipment or special equipment, depending on the circumstances. In this regard, the existence of a training policy for contractors and personnel is essential to the development of any new technique. The development of this training policy should preferably involve consultation with a country where the technique is used in the form of seminars, training periods, and monitoring of works in progress.

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Construction of Farm-to-Market Concrete Roads in Guipuzcoa, Spain

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The historical territory of Guipuzcoa, which belongs to the Basque Autonomous Community, is uniquely characterized by its rural settlements, the so-called caseríos, which are small family farms that perform a great deal of the agricultural activity in this Spanish province. Most of the caseríos are located on the mountainsides, because a great part of the narrow valleys, where the population in Guipuzcoa has developed, is occupied by industries and urban centers. Rural settlements are consequently scattered in isolated farmsteads or small hamlets to which access is difficult. In 1982, the Basque Autonomous Community Government and Guipuzcoan authorities therefore established a Plan for Rural Infrastructure Improvement, including technical aid and financial support for the construction or rehabilitation of rural roads. As part of this plan about 600 km of these roads were surfaced between 1982 and 1985, which represents about 50 percent of the total existing network. Both rigid and flexible pavements were used in road construction. Asphalt pavements are always constructed by contractors, but a traditional procedure of communal work is employed to construct concrete roads. This procedure is called "auzolan" in Basque, which means unpaid work performed to fulfill communal needs. Reductions in construction costs of over 20 percent have been achieved through the use of the auzolan work organization. Maintenance costs are also lower when compared with those of flexible pavements, because any repairs that are needed are normally performed by the farmers themselves. Therefore, the number of municipal authorities that are choosing to build concrete roads is steadily increasing.

The use of concrete in the construction of rural roads in the Spanish province of Guipuzcoa is described. This technique has proved to be a suitable solution to the problem of linking urban centers with the small farming units, or caserios. Substantial savings have been obtained by using communal labor methods to construct these concrete roads.

CONDITIONS IN GUIPUZCOA

Guipuzcoa is the smallest province (1997 km^2) in mainland Spain, although it is also one of the most densely populated. The population was 717,372, or 359 people/km^2, according to the 1979 census. Situated in the north of Spain, Guipuzcoa is bordered on one side by the Cantabrian Sea. Together with the provinces of Biscay and Alava, it forms part of the Basque Autonomous Community.

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Guipuzcoa is a very mountainous province that can be divided into two parts, depending on the altimetric distribution: the coast, with low altitudes that are generally below 400 m, and the interior, which gradually gains in altitude as the distance from the coast increases, and reaches elevations of above 1500 m at some peaks.

Its topography has a direct influence on the courses of its rivers. These rivers run perpendicular to the mountain reliefs, which they cross in very narrow, steeply banked valleys.

From a meteorological point of view, Guipuzcoa has a maritime climate. Average temperatures in the capital of San Sebastian, which is situated on the coast, vary between 8 and 19°C. The annual rainfall is over 1000 mm in the whole provincial territory.

All of these factors have conditioned the population of the province, because most human settlements are located on the coast and along the river valleys. Other regions are extremely underdeveloped, not only in demographic terms but also in terms of such factors as communications and the presence of industry.

The agricultural sector is not very significant in the Guipuzcoan economy; most economic activity is industrial. One of the components of industry, for historical reasons, is the metallurgical sector, which employs almost half of the active population. This type of company is usually established in the vicinity of important communications, such as ports, roads, and railways. For this reason, the topography of Guipuzcoa has led the majority of these industries to locate in a series of corridors along the river valleys, in addition to the metropolitan region of the capital, San Sebastian.

This combination of geographical determinants and industrial activity has resulted in a landscape typical of the superimposition of urban, industrial, and residential land in that it is characterized by congestion and a marked increase in the price of land. As a result of this phenomenon, farms have gradually retreated toward more inaccessible regions. Although agriculture has been modernized and farming methods and yields have improved, rural life has undergone an extensive decline parallel to the process of general development. Almost all agricultural production is based on the family-run caseríos.

A caserío basically consists of a house that simultaneously serves as a dwelling, a stable, and a storage facility; several head of cattle; arable land and pastures; and sometimes a lot of forest land. The family, which serves as both manager and farm hands, lives in and off the caserío itself. The family unit therefore comprises the basic component of social organization, an elementary community.

Farming systems vary, but are small in scale and above all inadequate for the type of activity involved. Most of the 11,700 caseríos that currently exist occupy an area of between 5 and 20 hectares. They are generally to be found to exist as isolated farms or in small settlements (Figure 1).

Although the caseríos are usually not situated very far from

FIGURE 1 Typical rural landscape in Guipuzcoa in which caseríos are shown.

developed regions, the fact that they are located in mountainous areas makes access to the towns difficult. This causes problems in both obtaining supplies of raw goods, such as feed and fertilizer, and collecting or marketing produce.

The cost of constructing or improving rural roads is high because the farmsteads are scattered in location and the steep slopes of the land create a need for retaining walls and culverts. The economy of the farming community is generally too small to bear these costs.

The Government of the Basque Autonomous Community and Guipuzcoan authorities (provincial government or Deputation, and municipal councils) therefore developed a Plan for Rural Infrastructure Improvement in 1982 as part of a Plan of Aid to the Agricultural Sector. This plan included the technical and financial aid needed for the construction and rehabilitation of rural roads. It enabled about 600 km of rural roads, or approximately 50 percent of the existing network, to be surfaced between 1982 and 1985.

LOW-VOLUME ROAD TYPES

The pavements of these rural roads are an average of 3 m wide. The traffic volume is low in all cases. The average daily traffic does not usually exceed 50 vehicles. However, many different types of vehicles use rural roads, from private cars to six-wheel lorries that carry logs from forest developments.

Three types of pavement are commonly used: a triple surface dressing on a granular base course, an asphalt concrete on a granular base course, and concrete. Concrete pavement is usually laid directly on the subgrade, without doing anything more than stripping the topsoil. However, a layer of granular material is sometimes laid to shape and fill the voids to ensure that the thickness of the concrete slabs, which usually varies between 10 and 15 cm, is as uniform as possible. The lengths of rural roads that were paved by each of the methods mentioned earlier over the period 1982 to 1985 are given in Table 1.

The municipal council within whose boundaries the rural road lies is responsible for deciding which type of pavement to use. As can be seen in Table 1, concrete has gradually come to prevail over the other alternatives. One of the reasons for this is that concrete roads are simple to build. Contractors are needed during the entire construction process on asphalt-paved rural roads. However, concrete pavements are constructed by using a method of communal labor that is a tradition in the rural Basque communities. This communal labor is called auzolan, which is a Basque word that means unpaid work performed to fulfill the needs of a community. This work can take such forms as road improvement, water supply, or repair to farms or chapels damaged by fire.

The farmers who use the rural road clear and shape the soil; spread and compact the granular material, if it is to be used; build ditches and culverts; and lay the concrete pavement. If they have no experience in handling concrete, a skilled worker is contracted to oversee the work.

The concrete is always prepared at a mixing plant (Figure 2). A proportion of 250 to 300 kg/m^3 of cement is used to obtain a characteristic compressive strength of 15 MPa at 28 days. Lime aggregate, the most abundant aggregate in Guipuzcoa, is normally used.

A single layer of plain concrete is usually used for the pavement. A reinforcing mesh sometimes has been used, in which case the pavement is placed in two layers. Approximately 50 percent of the pavements have no joints. Joints with boards have been placed in the rest of the pavement and no subsequent sealing is performed.

The sides of the road are formed with wooden forms before the concrete is laid. Another form is placed across them to serve as a guide when grading and compacting the concrete, and it is moved along the road as needed (Figure 3). Vibrating beams have been used in a small number of cases.

No change is usually made in the alignment of these rural roads, although improvements may have been made. Consequently, slopes are sometimes extremely steep; gradient values sometimes reach 30 percent. In these cases, the pavement

TABLE 1 LENGTH OF RURAL ROADS PAVED FROM 1982 TO 1985

Year	Triple Dressing	Asphalt Concrete	Concrete	Other Materials
1982	77 611 m	17 338 m	28 241 m	11 070 m
	(254,642 ft)	(56,886 ft)	(92,659 ft)	(36,321 ft)
1983	64 312 m	39 173 m	57 833 m	--
	(211,008 ft)	(128,527 ft)	(189,750 ft)	
1984	33 466 m	36 643 m	53 421 m	300 m
	(109,802 ft)	(120,226 ft)	(175,274 ft)	(984 ft)
1985	10 769 m	23 144 m	72 848 m	850 m
	(35,333 ft)	(75,935 ft)	(239,014 ft)	(2,789 ft)
Total	186,158 m	116,298 m	212,343 m	12 220 m
1982/85	(610,784 ft)	(381,574 ft)	(696,697 ft)	(40,094 ft)
(%)	35.3	22.1	40.3	2.3

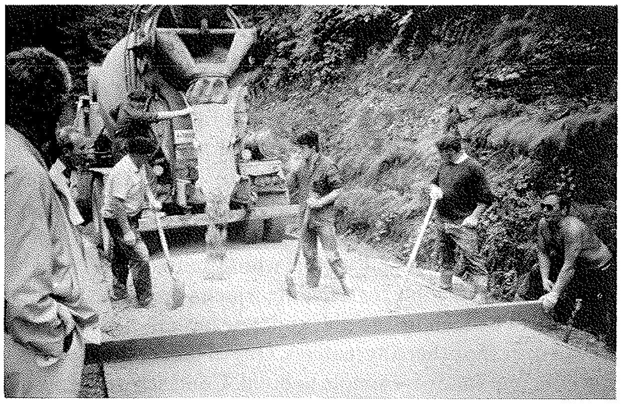

FIGURE 2 Supply of concrete for the construction of a rural road.

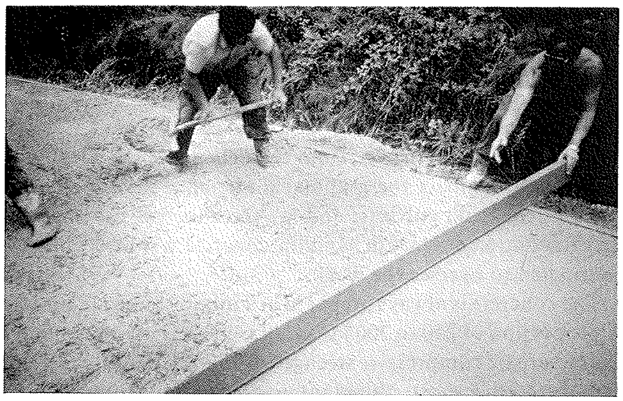

FIGURE 3 Compaction of concrete.

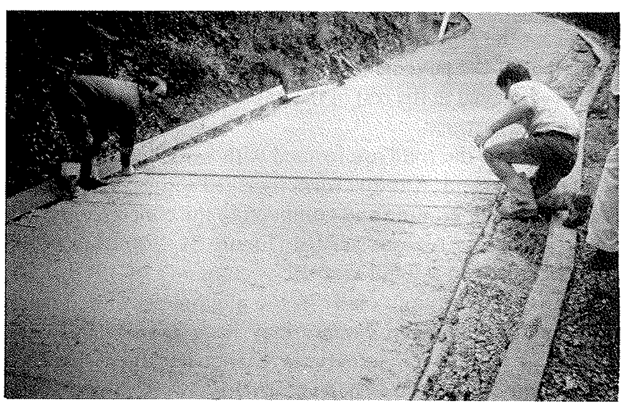

FIGURE 4 Applying grooves to freshly laid concrete in regions with steep gradients.

is usually given a transverse texture in the form of deep grooves that are made by combing the newly laid concrete with a metal rod (Figure 4).

In order to protect the edges of the pavement, a soft shoulder of one type or another is constructed of soil or granular material. The work is always performed under the supervision of engineers from the Deputation who are responsible for the design of the road.

Not only are concrete rural roads easier to construct than other types of pavement, but costs can be reduced by at least 20 percent through the use of the auzolan system in which the cost of labor is not added to the cost of materials.

CASE STUDY OF A CONCRETE LOW-VOLUME ROAD

The construction costs of a rural road 1500 m (4,920 ft) long that was built in 1985 are given in Table 2. The road is 3 m (9.8 ft) wide and the pavement is 15 cm (6 in) thick. The figures in the table were calculated on the basis of 1 U.S. dollar = 150 pesetas. The breakdown of the hours of unpaid labor is as follows:

Item	*Hours of Labor*
Clearing of site	682.5
Laying of pipes	190.0
Concrete pavement	1,329.5
Ditches and berms	2,154.0
Sign placing	8.0
Total	4,464.0

If an average value of 1 hour of labor at \$3.70 is estimated, the auzolan system saved \$16,147, or 25 percent, of the total cost of the work. This represents \$10.80/m of rural road or \$3.60/m^2.

In terms of the construction of the concrete pavement itself, the auzolan system saved \$4,920, or almost 15 percent of the cost of the pavement, and 9.4 percent of the total work cost. These savings could not have been attained with any other type of pavement. In addition, a cost of \$3.30/m (\$1/ft) of pavement was saved, and \$1.10/m^2 (\$0.10/ft^2) was saved.

The concreting of the pavement took 18 days, which corresponds to an average daily rate of 83 m (272 ft). The labor needed amounted to 0.9 hrs/m (0.27 hrs/ft). These outputs are within the normal range for this type of work.

TABLE 2 BREAKDOWN OF 1985 CONSTRUCTION COSTS OF A CONCRETE RURAL ROAD

Item	Total Cost (\$U.S.)[a]	(%)
Cutting for road widening	1,897	3.62
Rockfill for soil retention	3,157	6.21
Clearing of site[b]	--	--
Excavation of ditch for water pipes	570	1.09
Laying of pipes	844	1.70
Granular base course	3,628	7.31
Concrete pavement	33,502	67.50
Ditches and berms	6,332	12.09
Sign placing	245	0.47
Total	52,359	100.00

[a]The exchange rate in 1986 was 1 U.S. dollar = 150 pesetas.

[b]A total of 682 hrs of unpaid labor was used.

CONCLUSION

In addition to easier construction and lower costs, concrete roads in Guipuzcoa also have other advantages. Maintenance costs are lower because repairs can easily be made by the farmers themselves. Traffic safety is increased because a nonskid texture can be easily applied to the pavement. This is a great concern in regions with a topography and climate similar to Guipuzcoa. It is therefore not surprising to see that the statistics indicate that an increasing number of municipal authorities are choosing to construct concrete roads according to the auzolan system.

The Paving of Low-Volume Roads in Spain With Roller-Compacted Concrete

CARLOS JOFRÉ, ALEJANDRO JOSA, AND FERMIN MOLINA

The technique of paving low-volume roads with roller-compacted concrete (RCC) has been used for 17 years in northeastern Spain. The construction costs of this technique are economically favorable when compared to other structurally equivalent alternatives. Maintenance costs are also lower and the pavement is highly durable because the strength of roller-compacted concrete is similar to that of a conventional vibrated concrete. These are some of the reasons that justify its use. The advantages of the use of concrete as a paving material can be summarized as follows: conventional machinery can be used that is not specific to concrete; the newly paved surface can usually be traveled on immediately; and the materials are very economical because binders with a high fly ash content can be used. About 4 million m^2 of low-volume roads have been paved using this technology. An experimental study was undertaken in which tests were performed on core samples to check the performance of these RCC pavements during their years of service. The result is a detailed file on the performance and current condition of many of them.

Low-volume roads have generally received little attention in Spain in terms of their typology, design, construction, and maintenance. The central government's attention has instead been directed toward the main network, which has gradually been subjected to greater levels of traffic (*1*). However, data from the late 1970s indicated that the length of the network of Spanish roads with an average daily traffic (ADT) below 250 was 262 000 km (160,000 mi), or 84 percent of the total (*2*). This figure provides some indication of the importance of this secondary network to the country's communications. It forms an essential infrastructure for access to and communication between population centers, agricultural concerns, factories, and mines.

The characteristics of low-volume roads and the extent of their use require that cheap pavements be constructed that require little maintenance. Up to the present date the solution usually chosen has been that of a granular base course with successive surface dressings. This has resulted in pavements that are very cheap to construct but very expensive in the long run because of the need for frequent maintenance. Concrete pavements began to be used as an alternative. Contributing factors include the rise in the price of oil products during the 1970s and the acceptance of the partial replacement of cement (fly ash, slag, etc.), which reduces RCC costs and makes it competitive with alternatives. Concrete pavements also last longer and can go without maintenance for long periods of time, which leads to further savings in cost.

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The use of RCC in low-volume roads in Spain for the past 17 years represents a special case in the use of concrete pavements (*3*). The characteristics, behavior, and advantages of this technique are described in the following sections (Figure 1).

CHARACTERISTICS OF ROLLER-COMPACTED CONCRETE

Roller-compacted concrete is a uniform mix of aggregates, binder, and a small quantity of water that is laid and compacted by a roller, not by vibration (*4*). Its structure is similar to a cement-treated base, but its cement content and strength make it behave more like a traditional concrete. Its low water content does not allow it to be compacted by conventional concrete vibrators; heavy vibrating rollers are needed to compact it.

As a result of compaction, the aggregate skeleton attains a stability that allows the pavement to be opened to traffic immediately. Roller-compacted concrete at first behaves as a granular untreated base. For this reason, high short-term strength is not necessary, provided sufficient long-term strength is achieved, and the concrete does not deteriorate in the short term.

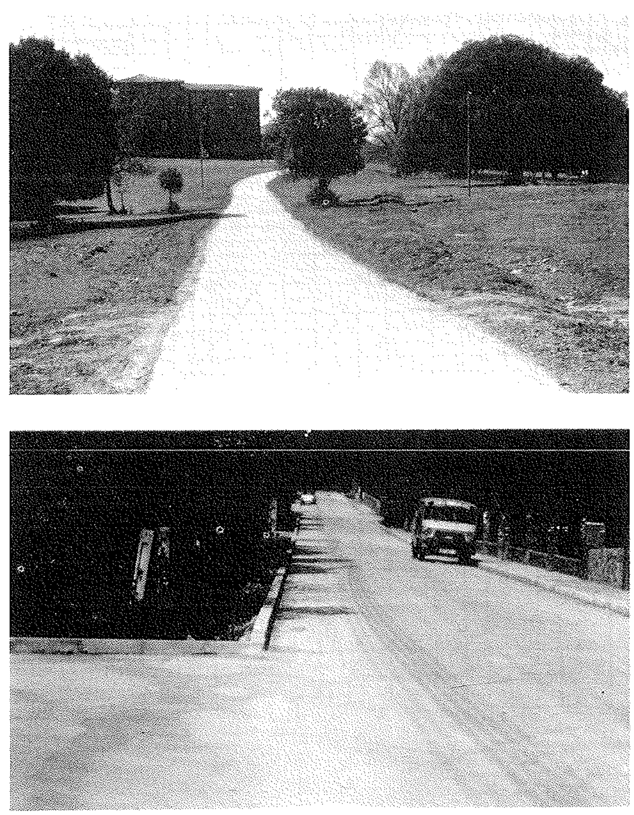

FIGURE 1 Low-volume, roller-compacted concrete roads in Spain.

Roller-compacted concrete is currently being used in dams and pavements. It has been used on roads with heavy traffic and on rural roads, and has a special application as an overlay, because traffic can be allowed onto it immediately (*5, 6*).

Materials

Aggregates

Grading limits established by practice are used to ascertain the grading of the aggregate. An example of the grading limits, including binder, that are provided in the Spanish specifications for RCC is shown in Figure 2 (*7*). It is particularly important to avoid an excess of the fraction passing the No. 200 sieve, which could cause surface depressions during compaction.

A maximum aggregate size of 20 mm (0.8 in) is specified for main roads; however, aggregate sizes of up to 38 mm (1.5 in) have been used on low-volume roads (*6*). A higher risk of segregation exists when high maximum aggregate gradings are used and special precautions have to be taken to avoid it.

Round natural aggregate and crushed stone have both been used. In the case of the former, compaction is easier but the resulting bearing capacity of the aggregate skeleton is reduced. In the case of the latter, compaction is more difficult but the bearing capacity is greater. At least 66 percent of the aggregates should be crushed stone.

Water

Water content is adjusted to obtain maximum density on compaction when laying. The maximum density obtained through the Modified Proctor Test (MPT) is used as a reference for this. However, because the compaction energy employed in the MPT is different from that applied on site, a difference exists between the optimum moisture content in the field and in the laboratory. In accordance with the experience to date, the field moisture content should be approximately 1 percent less than that determined in the MPT. However, it is advisable to err on the moist side of the moisture-density curve if the pavement is to be textured.

The customary values for water content in low-volume roads are 4.5 to 6.5 percent by weight, which implies a water-cement ratio of 0.36 to 0.42.

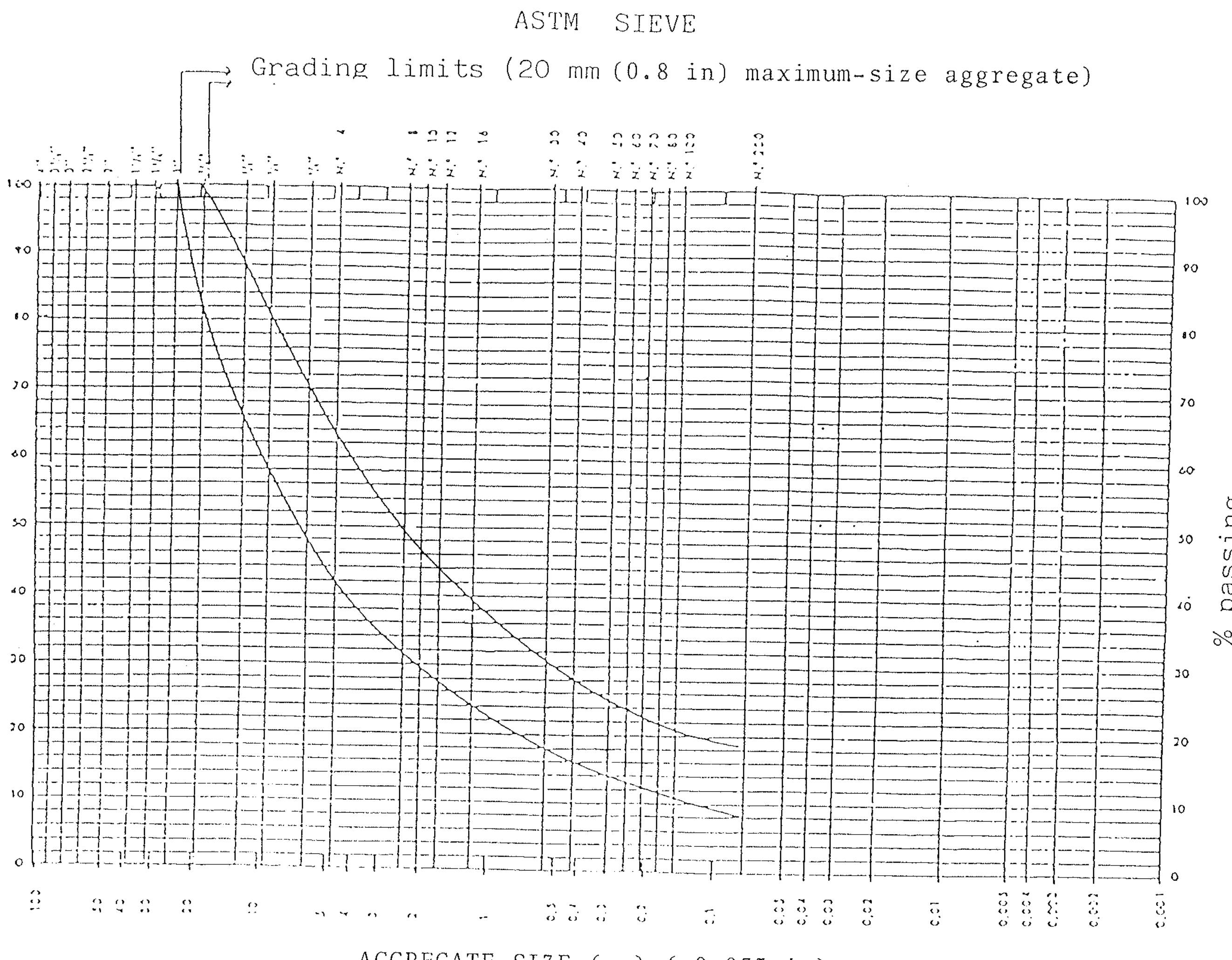

FIGURE 2 Grading limits specified for roller-compacted concrete.

Binder

The methods used to lay RCC are such that binders in which the cement is partially replaced with fly ash and slag can be used (*8*). Although these binders may also be used in traditional concrete pavements when the speed at which the road can be opened to traffic is not critical (*9*), they are especially suitable for RCC, because their slower setting speed facilitates compaction operations.

The typical evolution in strength is shown in Figure 3. In this example, the cement contained 50 percent ASTM Class F fly ash. The strength increased considerably between 28 days and 3 months; design strengths beyond 28 days can therefore be specified.

The usual quantities of binder range from 230 to 375 kg/m^3 (390 to 630 lb/yd^3), 10 to 12 percent by weight. The proportions must be determined by prior tests for each binder, and the quantity of water required should be based on the MPT. It is very important to check the sensitivity of the strength to variations in moisture content and density in these tests to determine the risk of lack of strength as a result of these variations.

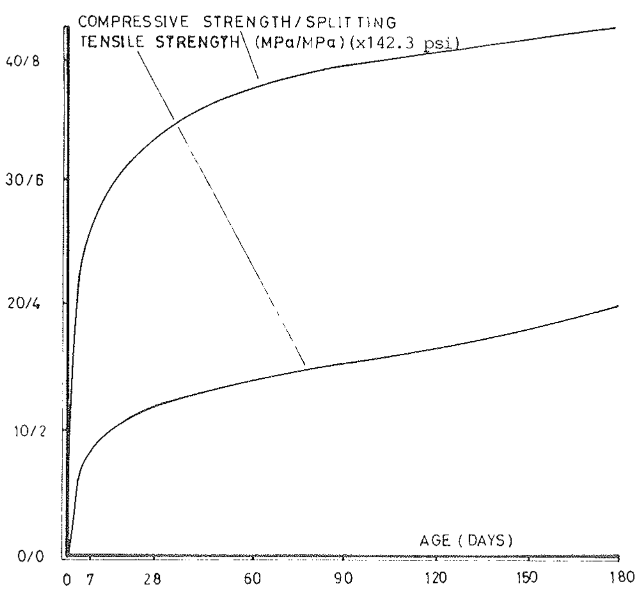

FIGURE 3 Compressive strength and splitting tensile strength versus age for roller-compacted concrete with 7 percent fly ash and 7 percent cement used in an overlay project.

Admixtures

It is sometimes advisable to use admixtures, particularly plasticizers that allow the quantity of water to be reduced and consequently increase strength. Admixtures can also be used that retard the setting of concrete. These admixtures are basically necessary when the pavement is laid in half-widths to avoid the occurrence of undesirable longitudinal joints.

Thickness Design of RCC Pavements

In the absence of a specific design method for RCC pavements, it is currently admitted that the same pavement thicknesses can be adopted for RCC or an ordinary vibrated concrete with a similar strength. The strength of RCC is controlled by means of splitting tensile tests using cylindrical samples 15 cm (6 in) in diameter and 18 cm (7 in) high, because it is very difficult to make true prismatic samples.

The testing age depends on the type of binder used, in accordance with the requirements of the specifications, and may be at 28, 56, or 90 days. The recommended minimum strength for low-volume roads is 2.8 MPa (400 psi).

A 20-cm-thick (8-in-thick) soil-cement subbase under the RCC layer is specified for main roads, whereas granular, untreated subbases are used for roads with medium levels of traffic. Roller-compacted concrete pavements are usually placed directly on the subgrade of low-volume roads; only a top layer of organic soil is eliminated, when necessary.

Shoulders are not normally provided on low-volume rural roads. This lack of lateral constraint results in some reduction in the degree of compaction obtained alongside the pavement. Curbs are usually placed on urban streets before the RCC is spread (Figure 4).

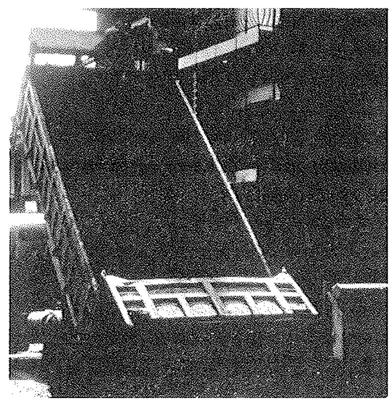

FIGURE 4 Hauling concrete with dump trucks.

Laying

Production

Either continuous plants or batch plants have been used to mix the RCC. Because the percentage of binder is similar to that of a vibrated concrete, the continuous plant must be able to proportion these quantities. In either case, a weight control is essential, particularly for the binder, to ensure that the material proportions are correct.

Transport

The RCC is transported in dump trucks (Figure 4). Desiccation of the material must be avoided, which makes the use of protective canvases essential if the distance is long or the temperature is high. The distance from the plant to the site should be limited for this reason.

Spreading

Spreading is usually accomplished with a motor grader (Figure 5), which can easily be adapted to complex geometrical shapes.

FIGURE 5 Spreading with motor grader.

Compaction

Vibrating rollers are normally used in combination with rubber-tired rollers for compaction (Figure 6 and 7). In the first stages of compaction, vibrating rollers are used without vibration to ensure that the surface is even. Rubber-tired rollers are used in the final stages to seal the surface of the concrete. The number of passes the vibrating roller makes must be sufficient to ensure that the density of the material is correct (97 percent of the MPT). The rubber-tired roller is not needed in cases in which the texture is created in the concrete itself.

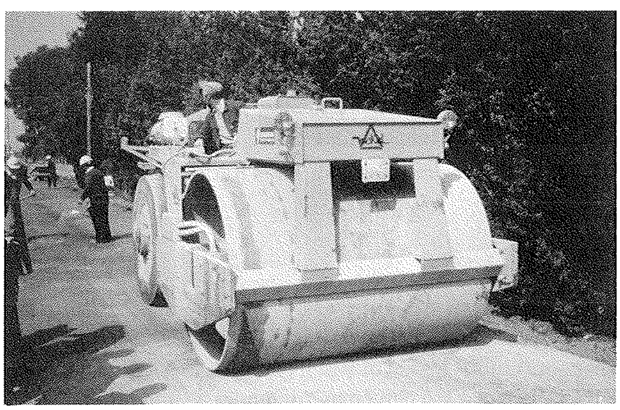

FIGURE 6 Compaction with smooth-drum roller.

FIGURE 7 Compaction with rubber-tired roller.

Surface Finishing and Curing

Two techniques are normally used to finish and cure the surface. In the first technique, a coat of curing compound (asphaltic emulsion) is protected with sand for provisional traffic; this is followed by a double surface dressing. In the second technique, a cement slurry is sprayed on the fresh concrete; this slurry is then promptly troweled by a machine called a "helicopter" (Figure 8 and 9).

In the second technique, the water content should err on the moist side of the moisture-density curve. It is therefore necessary to use aggregates that basically consist of crushed stone to be able to compact the material with this higher moisture content.

Sawing of Joints

The provision of joints in RCC pavements is a subject of controversy. If joints are to be created, they must be sawed. However, in many cases joints are not actually created, but are allowed to occur spontaneously. The sawing time is generally less critical than it is for vibrated concrete; in any case, it depends on the type of binder used and the temperature involved. Joints or cracks remain unsealed in all cases.

Other Operations

Other operations worthy of mention are the end-of-day joints (Figure 10), the elimination of longitudinal joints in the case of pavements that were laid in half-widths, and the moistening of the surface (Figure 11). This last operation is very important to prevent the surface of the concrete from becoming desiccated.

Advantages and Disadvantages

The advantages of roller-compacted concrete are as follows:

- No special machinery is necessary to lay concrete pavements (slip-form pavers) and the production rate is much better than laying forms by hand and using a vibrating beam (100 to 200 m/day or 330 to 655 ft/day).
- Roller-compacted concrete can be used as an overlay while the pavement is open to traffic in cases in which traffic cannot actually be stopped.

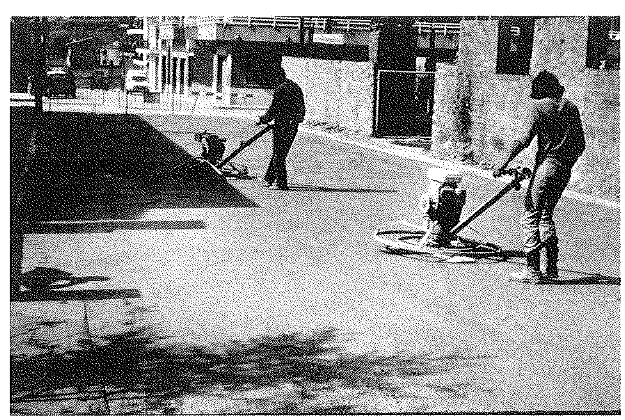

FIGURE 8 Texturing with power trowels.

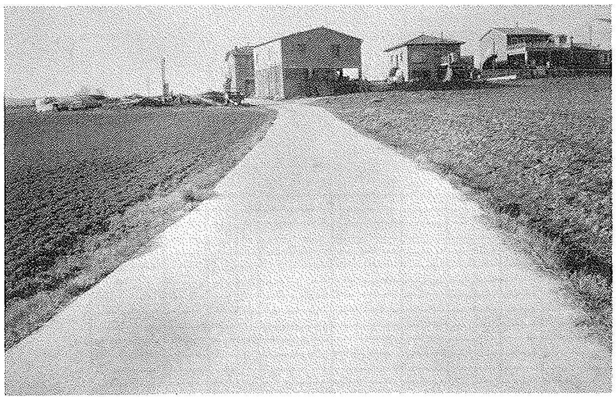

FIGURE 9 Texture obtained through use of power trowels.

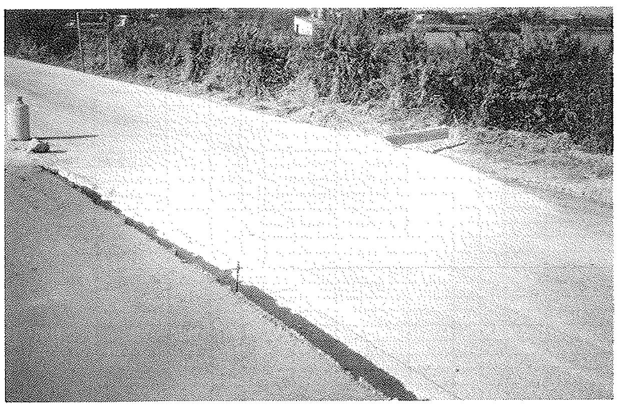

FIGURE 10 Sand wedge at construction joint.

FIGURE 11 Spraying of concrete surface to prevent moisture loss.

- Energy consumption is reduced when the concrete has a high content of additional binders such as fly ash.
- Less hydraulic shrinkage occurs as a result of the smaller water-cement ratio. This allows joints to be set further apart (up to 15 m or 50 ft).

One of the disadvantages of roller-compacted concrete is that it has a greater sensitivity to changes in humidity and density. Another disadvantage is that the road is rougher than it would be if slip-form pavers were used. This problem can be solved on main roads by using pavers instead of motor graders and, if necessary, placing a thin layer of asphalt concrete on top. This is not a problem for minor roads because there is no high-speed traffic. In any event, the skill of the motor grader operator is of paramount importance.

EXPERIENCE WITH RCC ON LOW-VOLUME ROADS IN SPAIN

The first known pavements that were built with RCC in Spain date from 1969 and 1970. They were constructed in the northeastern section of the country in small towns, housing states, and country roads. A single company constructed most of these pavements and continues to do so. A total of about 4 000 000 m^2 (4,800,000 yd^2) of pavement were laid. The following method was used:

- The material was produced in plants similar to those used for cement-treated bases.
- It was transported in dump trucks within a delivery radius that was limited to 30 to 35 km (20 to 22 mi).
- It was then spread with a motor grader.
- It was first compacted by a vibrating roller without vibration; four or five passes were made then with vibration.
- It was textured by the addition of cement slurry and mechanical troweling by a "helicopter."
- The joints were sawed every 10 to 12 m (33 to 39 ft) within 24 or 48 hrs after the material was spread.

Both the proportions used and the method of laying have developed over the years to reach a stage of technology that is very well-adapted to low-volume roads. Crushed limestone aggregates have generally been used. The mix has usually been based on three different sizes of aggregate to obtain the final grading curve. The binder that is currently being used is a mixture of Category P-450 cement and ASTM Class C fly ash. The proportions are as follows (Figure 12):

Gravel 10 to 30 mm (0.4 to 1.2 in)	230 kg/m^3 (390 lb/yd^3)
Gravel 0 to 10 mm (0 to 0.4 in)	1630 kg/m^3 (2,750 lb/yd^3)
Sand 0 to 5 mm (0 to 0.2 in)	240 kg/m^3 (400 lb/yd^3)
Category P-450 cement	170 kg/m^3 (290 lb/yd^3)
ASTM Class C fly ash	90 kg/m^3 (150 lb/yd^3)
Water	110 L/m^3 (185 lb/yd^3)

Normal structural sections have consisted of 15 cm (6 in) of concrete, or 20 cm (8 in) in the case of greater traffic levels, that were laid directly on the subgrade or on top of 15 cm (6 in) of granular subbase, in the case of low-quality subgrades. Pavements in rural roads have been an average of 3.5 m (11.5 ft) wide. It should be noted that the maximum single axle weight allowed in Spain is 130 kN (28 kips). The characteristic compressive strength of the concrete used at 28 days ranged from 20 to 25 MPa (2,850 to 3,560 psi).

Very few controls have been applied to RCC construction. The experience of the workers who laid the pavements has led to satisfactory results; most of the pavements have presented no problems, even after 17 years in service.

The reasons that led to the use of this technology were its competitiveness with other solutions in terms of cost (about 1,000 pesetas/m^2 or \$6.7/m^2 in 1985), and the fact that only a low level of maintenance is needed for a long period of time. The

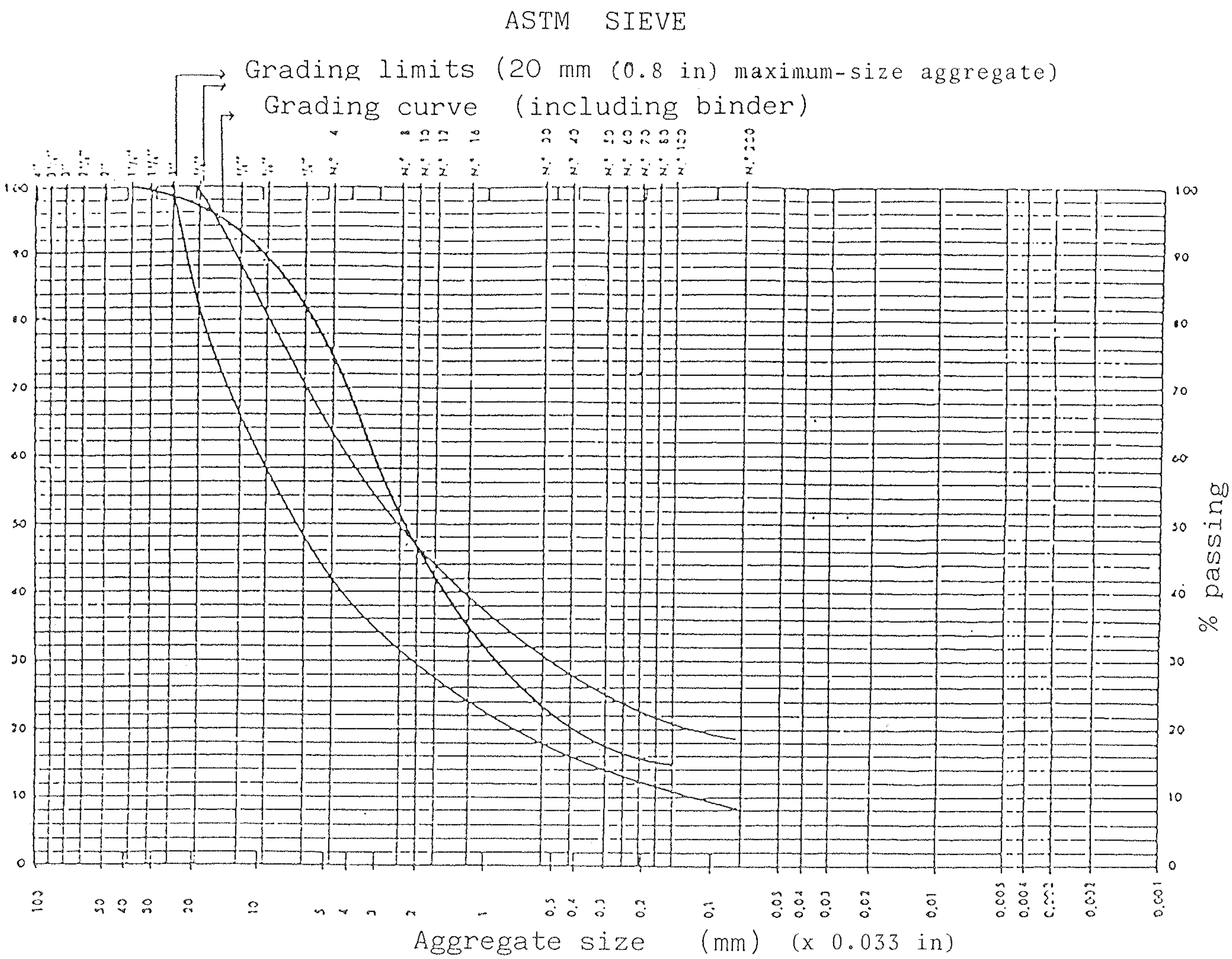

FIGURE 12 Grading curve used, including binder and grading limits.

latter is very important for local or private groups that do not have the means to perform such maintenance.

Several administrations have begun to use the RCC technique in two different ways. In the first method, the concrete has a water content that is on the moist side of the MPT and the surface is textured by a mechanical trowel called a "helicopter." This method is advantageous in that the surface texture is created in the concrete itself without the use of asphalt products. However, pavements textured in this manner cannot be opened to traffic immediately; a delay from 1 to 2 days has to be respected in this case.

In the second method, the concrete has a water content that is on the dry side of the MPT, and it is cured by spreading an asphalt emulsion on it. The surface is textured with a double surface dressing. This method is advantageous in that traffic can travel over the pavement immediately.

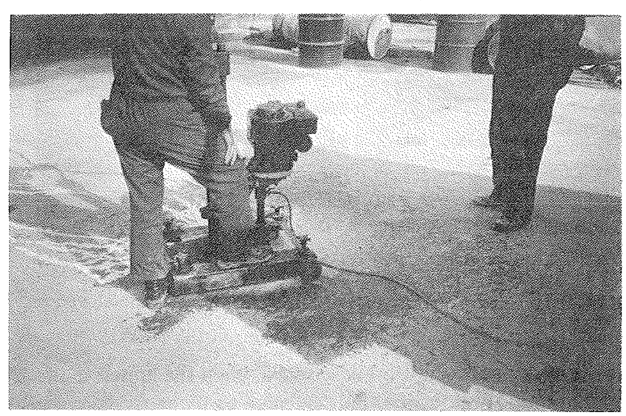

FIGURE 13 Extraction of cores.

STUDIES PERFORMED

As a result of the interest shown in this technique and the favorable results obtained from using it, it was decided in 1985 to extract cores from several of the completed projects (Figure 13). The aim was to check the current condition of the pavement and to obtain data in regard to the strength and density of the concrete. Twenty projects were chosen that were completed from 1969 to 1984; these included projects with slab thicknesses of 15 cm (6 in) and 20 cm (8 in).

The condition of each project was visually inspected and four cores (Figure 14) were extracted. Two of these cores were tested by compression (Figure 15) and two by the splitting tensile test (Figure 16). The densities of 20 cores were measured. The actual thickness of the slab was measured in each case and the condition of the extracted core was visually inspected to study such characteristics as porosity and degree of compaction.

The following conclusions were drawn from the findings:

- Although the results vary, the compressive strength was above 25 MPa (3,550 psi) in 80 percent of the cores and the splitting tensile strength was above 2.8 MPa (400 psi) (Figures 17 and 18). The variation in the results can be attributed to the limited control during construction. However, the strength rates were generally high and reached almost 60 MPa (8,500 psi).
- As was expected, a link existed between the density and strength of the cores; the greater the density, the greater the strength (Figure 19).
- As was also expected, the projects in the poorest visual condition (most cracking) were also those with lower strength, lower density, and the most porous cores, and vice versa.

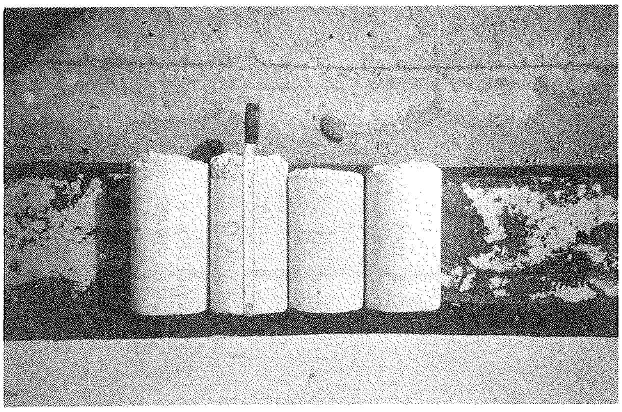

FIGURE 14 Cores extracted from a project.

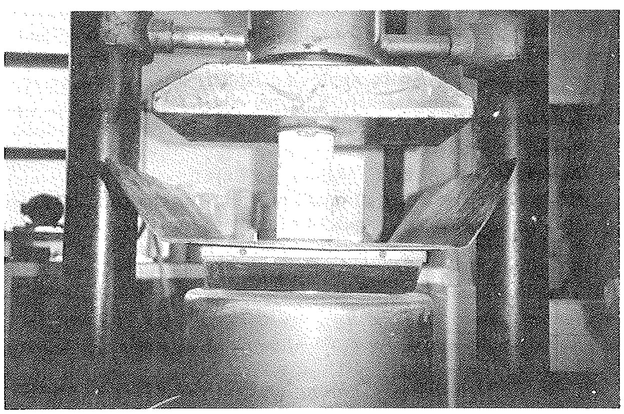

FIGURE 15 Compressive strength test.

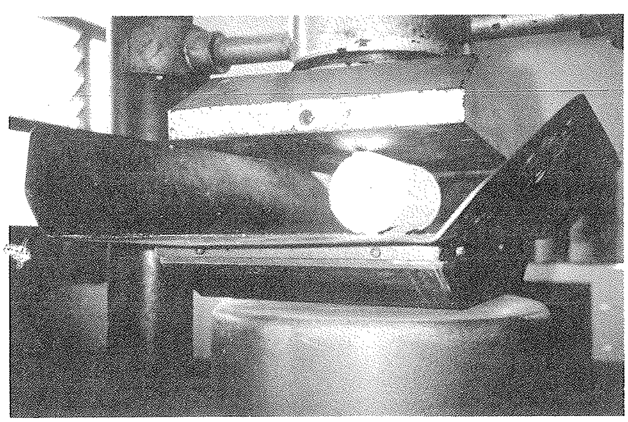

FIGURE 16 Splitting tensile strength test.

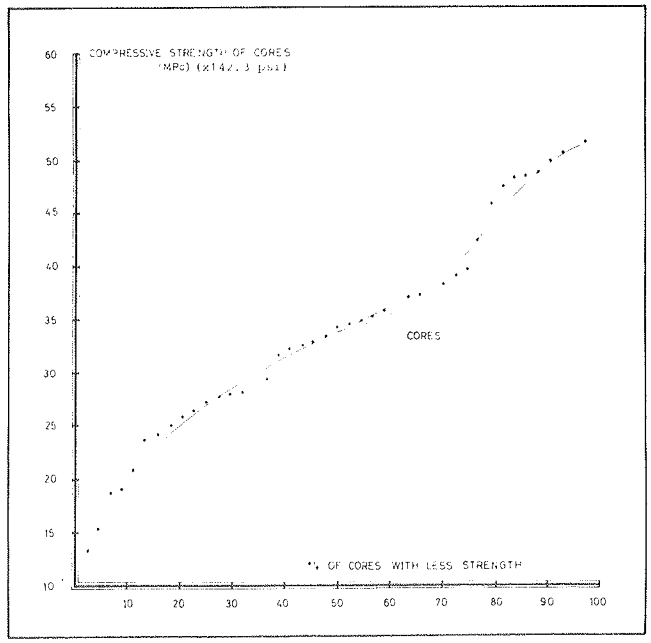

FIGURE 17 Compressive strength of cores versus percent of cores with less strength.

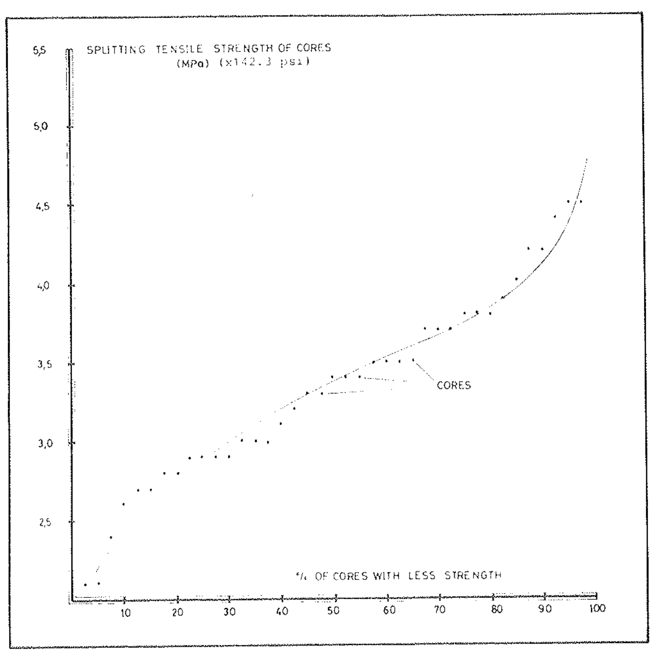

FIGURE 18 Splitting tensile strength of cores versus percent of cores with less strength.

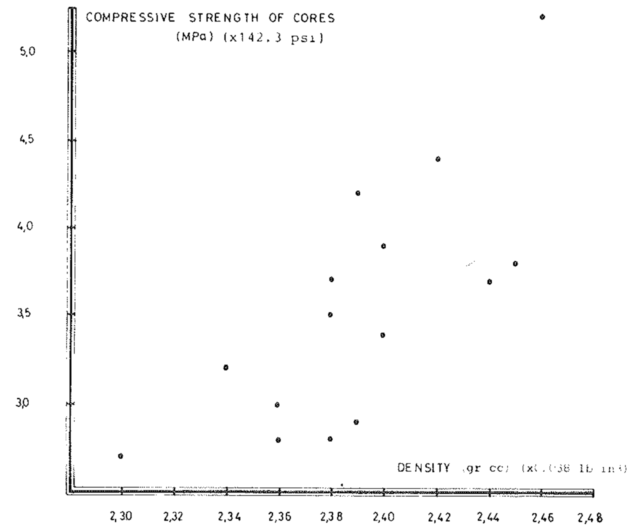

FIGURE 19 Compressive strength of cores versus density of cores.

- Thickness also varied; in some cases the slab thickness was either too little or too much. This can be attributed to defects in the leveling of the subgrade or subbase, among other things.
- Although an increase in strength was expected in older projects because of the increase in strength provided by the fly ash or slag in the binder, laying faults or variabilities in each project completely obscured this correlation (Figure 20).

CONCLUSIONS

The experience gained with the use of roller-compacted concrete on low-volume roads has demonstrated that this technology is highly appropriate. It meets the two conditions that are required of these pavements: competitiveness in construction cost and low maintenance over a long period of time. It appears that both conditions have been met in practice, and studies have confirmed that such roads have performed well.

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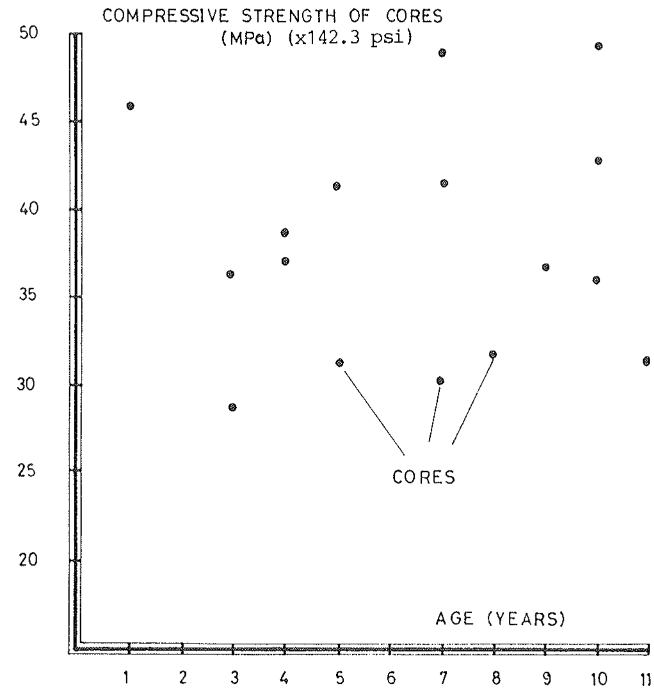

FIGURE 20 Compressive strength of cores versus age of cores.

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The Design of Low Water Stream Crossings

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When a bridge becomes obsolete, and the road must remain open to traffic, perhaps at a new location, a low-cost alternative may be to replace it with a low water stream crossing. A low water stream crossing consists of a series of culverts that are deliberately designed so that the crossing is at a low grade and stream flow at high water frequently overtops the grade. A design manual for low water stream crossings was prepared for the Iowa Highway Research Board. A description is provided of the major steps and considerations in the design of a low water stream crossing. The decision to build a low water stream crossing is based on the road classification. A primitive road is an excellent candidate. The first step is to select the frequency of overtopping that can be tolerated and then calculate a discharge Q_e. A series of pipes are selected with this overtopping discharge value in mind to minimize the roadway fill over the stream. The design procedures offer criteria for the grade line design and for the final cross-section of the roadway. General construction details and guidelines for the selection of materials and final signing are also presented.

A low water stream crossing (LWSC) is a street or road that crosses a stream; the flow of storm water in the stream periodically overtops the roadway. The roadway may frequently be flooded, in which case the road must be closed to vehicular traffic during the higher stages of stream flow. Low water stream crossings are grouped into two main types in this discussion: unvented fords and vented fords.

An unvented ford is a roadway that crosses the stream without the use of any pipes (culverts). Low flows in the stream may pond and flow over the roadway if the stream flow is intermittent, or low flows may overtop the roadway most of the time. The early settlers of this nation located trails so that they would be able to cross streams at points where the streambed was hard and the water depth during relatively dry periods allowed for the passage of vehicles. A roadway can be built above minor streams except a channel must be provided near the center. On larger streams the ford may only consist of approach ramps that lead to a relatively stable stream bed.

A vented ford consists of a cross-section for the roadway above the stream bed, and a pipe or number of pipes under the roadway that will provide for low water stream flows without overtopping the roadway. High water will periodically flow over the roadway because the pipes are deliberately sized to be too small for all but the smallest flows.

Another type of LWSC is a low water bridge. A low water bridge is a flat-slab bridge deck at about the elevation of the adjacent stream banks, with a smooth cross-section that is designed in such a manner that high water will flow over the slab without damaging the slab bridge; when the water recedes, the bridge can be used immediately.

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GENERAL APPLICATIONS

An unvented ford is primitive. If the stream has a continuous flow of water, normal automobile traffic may encounter operational problems at the wet crossing. Four-wheel-drive and farm vehicles may not encounter problems, except during high water flows. Specialized access roads may consequently have to be built that are suitable for unvented fords. The unvented ford also requires considerable maintenance. Limited design criteria are available for this road type. The only application for unvented fords often is at intermittent streams that are dry for a significant portion of the year.

The vented ford LWSC, however, has numerous applications because the design can limit the flow over the roadway to a very few days of the year. If the closing of the road for short periods of time can be tolerated, the LWSC may offer significant savings over a culvert with a roadway fill designed to provide for a 25- or 50-yr discharge without overtopping.

A primitive road that serves only as a field access for local farmers is a classic example of an LWSC candidate. During good weather conditions, a well-designed vented ford can perform adequately for any traffic using the road. In fact, an LWSC might be superior to the typical obsolete bridge found at this site. This type of bridge might be a wooden structure that was built on a narrow roadway just after the turn of the century. Farmers using modern farm equipment even have problems with modern bridges. Bridges were not designed for farm equipment that commonly reach widths of 18 to 20 ft, and that in unique cases reach 28 ft with axle loads approaching 80,000 lbs. A farmer may be better served by an LWSC if vandals were to set fire to a bridge, or heavy equipment was to cause it to fail structurally.

During periods of dry weather, a primitive road is passable by most vehicles and the LWSC performs suitably. During periods of significant rainfall, the primitive road is only used by farm vehicles, and the closing of the LWSC does not inconvenience the general traveling public.

However, not all obsolete bridges are on a primitive road that serves only as a field access. Other potential locations for an LWSC in which a short loss of access can be tolerated are those that have a suitable alternate route, or detour, but that do not have residences with sole access over the LWSC, a critical school bus route, recreational use, or a critical mail route.

The size of the drainage area can also affect the decision of whether or not to use an LWSC. During high flows on a small watershed, flood waters rise and subside rapidly, whereas on a larger watershed, flood waters rise more slowly and flow over the LWSC for a longer time. It therefore may be tolerable to close a road for a short time as a result of an LWSC on a small watershed. However, it may not be tolerable to close a road for a longer period of time.

Traffic volume as a criterion for LWSC use can be misleading. Significant volumes of traffic indicate a user demand for that particular route. Closing an LWSC with relatively high traffic volumes temporarily increases user costs by diverting traffic to an alternate route. Another reason that is perhaps more significant is that a larger volume of traffic increases the probability that a user will take chances and cross a flooded LWSC when the road should be closed.

Surfacing or pavement type is not necessarily a criterion for LWSC locations. An unsurfaced road obviously indicates a route of lesser importance. In this case, periodic closing is probably of less concern to the user. However, a high-quality surfacing might indicate a high users' demand for improved facilities on an important route, and therefore a reason for providing a higher level of service.

An LWSC may in fact be applicable when used in combination with an existing obsolete bridge. Consider the situation of a wooden bridge with a substandard width and a lack of structural capability to handle farm equipment. If this bridge was posted so as to preclude all vehicles but automobiles, and a "shoo-fly" vented or unvented ford was provided adjacent to the bridge as shown in Figure 1, both types of users would be served. A situation in which the heavier types of equipment would be unable to use either type of crossing is infrequent.

GENERAL DESIGN CRITERIA

An overview is presented of the entire design process. Because each site is unique and has its own set of conditions, the following criteria and concepts should be viewed as general guidelines that can lead to a well-designed, safe crossing.

Components of an LWSC

An LWSC consists of several components: core materials, foreslope surface, roadway surface, pipes (if it is a vented ford), and cut-off walls or riprap to protect against stream erosion.

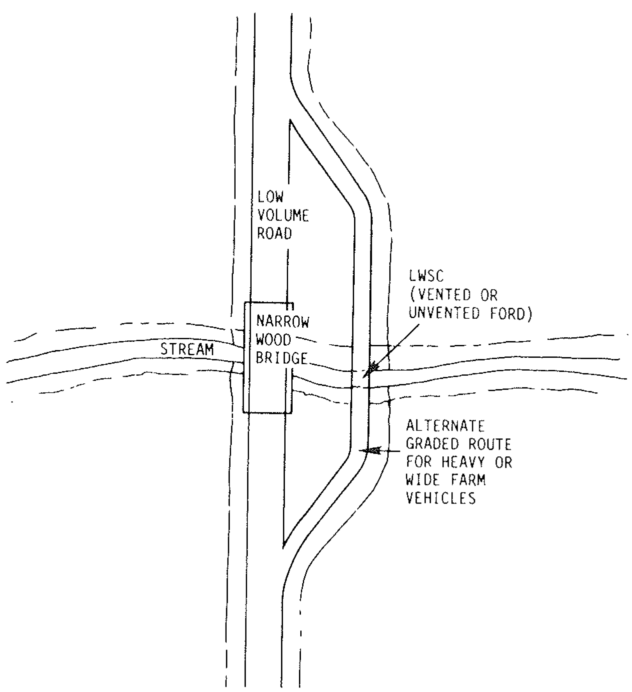

FIGURE 1 Combination obsolete bridge with alternate LWSC for farm equipment.

The core can consist of earth, sand, gravel, riprap, concrete, or a combination of these materials. Erosion protection for the foreslopes can consist of turf, riprap, soil cement, gabions, or concrete. The roadway surface can be composed of similar materials with the provision that a suitable riding surface be developed. The cost and availability of these materials vary from region to region; therefore, the exact composition of the core and surfacing depends on local conditions. Pipes can be shaped like circles, ovals, rectangles, or arches, and can be made of concrete, corrugated metal (CMP), or polyvinylchloride (PVC).

Protection against stream erosion can be provided by either cut-off walls or by armoring the steam bed. Cut-off walls can be constructed of either concrete or steel. The armoring could be in the form of riprap or gabions. The question of whether to use steel, concrete, or rock again depends on the local cost and availability of materials and equipment, such as a pile driver. These components are depicted in Figure 2.

Basic Steps in the Design Process

The general steps involved in the design of an LWSC are diagrammed in Figure 3. The first step is to analyze the location and all the factors that are involved in the decision to build an LWSC. The question of whether to use a ford or a vented ford depends on whether or not water over the road can be tolerated. In most cases an unvented LWSC will create problems as a result of having to close the road for significant periods of time, except in special cases.

The allowable overtopping duration and frequency is a function of local conditions that are unique to each site. Once the percentage of the probability of overtopping (and road closing) has been determined, the overtopping discharge (Q_e) can be calculated. The number and size of the pipe or pipes are then selected so that the head water depth for Q_e just reaches the lowest point in the roadway design.

The crossing grades and elevations are a function of the physical features of the channel and stream banks, and are related to the overtopping discharge headwater depth. The headwater depth and the vertical curve length for a given speed are checked, and the number and size of pipes are then adjusted accordingly.

The selection of material for the crossing foreslopes and the roadway surface is a function of the overtopping velocity and the tractive force, and could range from turf to concrete. The overtopping velocity of the Q overflowing the road is critical until tail water submerging occurs. The final step in the design is to provide protection against stream erosion and seepage.

THE DETERMINATION OF OVERTOPPING FREQUENCY AND DESIGN DISCHARGE

This basic step in the design process requires that a decision be made as to the percent of time in a year the LWSC may be closed; the overtopping discharge (Q_e) can then be calculated.

Overtopping Frequency

The selection of an exceedence probability percent is based on the conditions at the site. The need to have the road open is

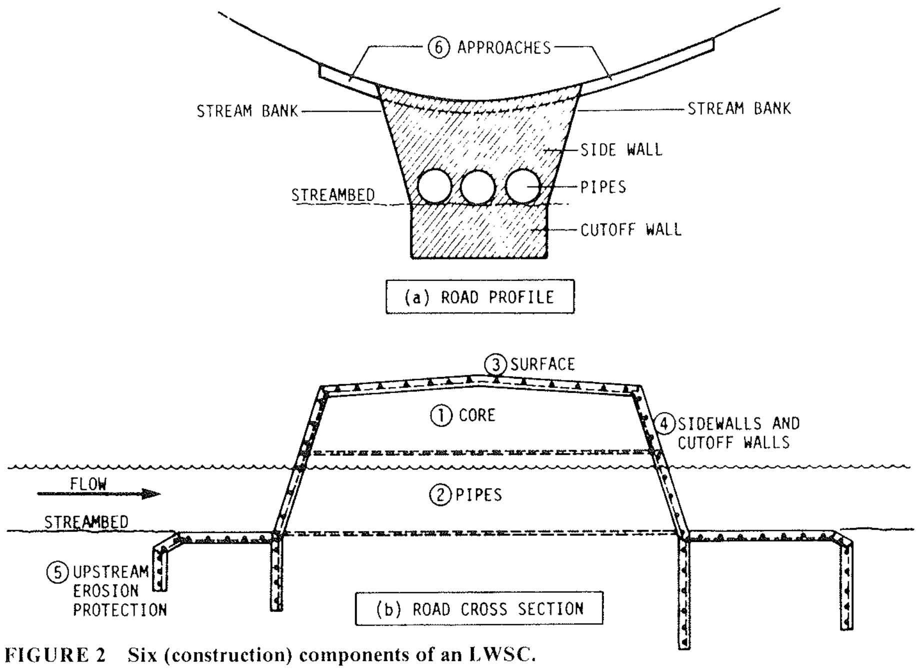

FIGURE 2 Six (construction) components of an LWSC.

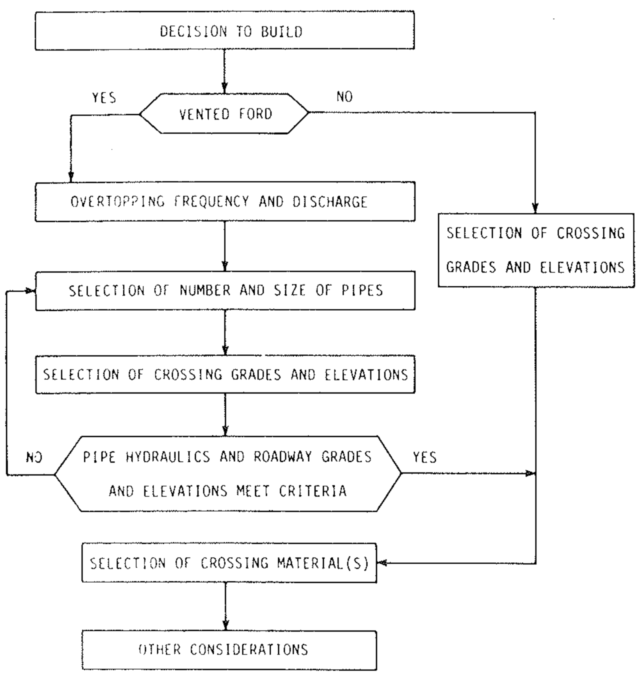

FIGURE 3 General design steps for a low water stream crossing.

based on the type and volume of traffic and the characteristics of the users. A farm field access road with no other traffic could be closed more frequently than a road that serves as access to a home or a school bus route.

A decision to use an exceedence probability of 10 percent would mean that water would flow over the road an average of about 37 days a year. The resulting design discharge would be $Q_{10\%}$. The selection of a design discharge of $Q_{2\%}$ would mean that water would flow over the road an average of one week of the year.

Overtopping Discharge

Once the exceedence probability is selected, the discharge in cubic ft for this probability can be determined by two methods.

If recorded data of daily discharges at the stream location where the LWSC is planned are available, a flow-duration curve can be prepared. This curve indicates the percent of time in which given rates of flow are equaled or exceeded. The curve is prepared by arranging the collected daily discharges in class intervals of ascending order of magnitude. The percent of time during which the flow was equal to or greater than the lower limit of each class is determined and the results are plotted as a flow-duration curve, as shown in Figure 4. The exceedence probability is selected and the discharge is determined from the curve.

Flow-duration information is more frequently needed at stream crossings where no recorded data are available. Low flow records are usually available from the U.S. Geological Survey for certain streams with gaging stations. In some states these data have been statistically analyzed on a regional basis and regression equations have been developed. The form used in Iowa is as follows:

$$Q_e = {}_aA^b$$

where

Q	=	discharge in ft^3,
e	=	exceedance probability expressed as a percentage,
A	=	drainage area in mi^2, and
a and b	=	regression coefficients peculiar to a particular similar region.

In a case in which no regional equations have been developed, the only technique available is to use adjacent flow-duration curves similar to that shown in Figure 4.

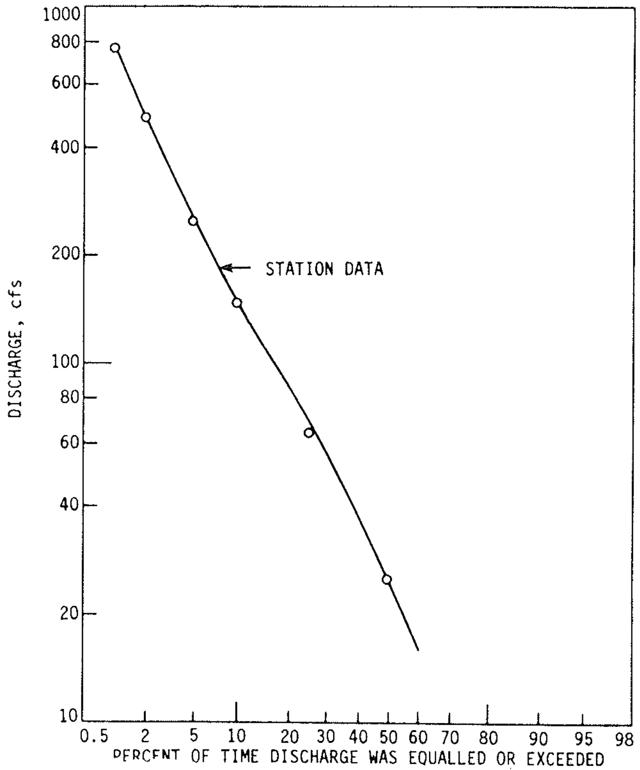

FIGURE 4 Duration curve of daily flow, Timber Creek near Marshalltown, Iowa, 1949-81.

THE DETERMINATION OF THE NUMBER AND SIZE OF PIPES

The determination of the number and size of pipes for a particular site is a trial-and-error process. Several items must be kept in mind: (a) the total width of pipes, including the spaces between them, must be less than the width of the existing channel; (b) the headwater depth controls the low point in the roadway; (c) the pipes may operate under either inlet control or outlet control; (d) pipe lengths are short, but differences in friction losses as a result of pipe material still could be significant; (e) a large difference between the low point in the roadway and the downstream water surface increases the erosion potential on the downstream foreslope; and (f) a large difference between the low point in the roadway and the stream bed increases the volume of material needed in the crossing and, therefore, its cost.

The information needed to determine pipe size is available in Herr and Bossy, "Hydraulic Charts for the Selection of Highway Culverts," *Hydraulic Engineering Circular 5*, GPO, 1964. This publication is commonly known as HEC Number 5 or Bulletin 5. Several combinations of pipe sizes and numbers should be selected for analysis. By using the appropriate chart in Bulletin 5, the headwater depth can then be determined for each combination in a manner similar to a culvert design procedure.

The trial-and-error process begins by determining headwater depths for the estimated overtopping discharge and assumed combinations of pipe material, number, and size operating under inlet control. The results are reviewed in light of the previously mentioned items and the several combinations are reduced to the few best alternatives. These alternatives are checked for outlet control and the final type, size, and number of pipes are selected. If the final low point in the roadway is higher than the calculated headwater depth as a result of roadway criteria, the possibility then exists that the number or size of pipes, or both, could be reduced; this should be checked.

ROADWAY GEOMETRICS

Low water stream crossings are designed for occasional overtopping with floodwater and consequently have an inherent vertical dip characteristic. The approach roadway is at or above the normal ground level on the stream banks, whereas the low point of the crossing may be much closer to the normal water flow surface than a normal culvert design.

This sudden dip in the vertical alignment is inconsistent with drivers' expectations of a public highway profile. Proper signing is essential to alert the driver to a condition that should not be traversed at the higher speeds associated with tangent alignments and flat grades.

The variables of concern in the design of the stream crossing profile are (a) the tangent grades, (b) the length of sag vertical curves, and (c) the length of crest vertical curves at the edges of the stream.

The Determination of Tangent Grades

The determination of tangent grade lines depends on the height of the stream banks, the slope of the terrain adjacent to the stream banks, and the amount of cut allowed into the stream bank. If minimal grading is desired, steep grades will result. A grade of 12 percent should generally provide a surface suitable for driving when wet and muddy, but only at very low speeds. This arbitrary maximum may in fact be increased without undue concern if the vehicles consist of farm equipment and four-wheel-drive automobiles and speeds are very low. Steep grades significantly increase the stopping distance and consequently reduce the allowable speed. However, flat grades that cause a cut-back into the stream bank can result in a maintenance problem. Mud and debris may be deposited by the recession of high water.

The Determination of Vertical Curve Lengths

A number of criteria are recognized in the design of a profile. The criterion of stopping sight distance (d) is usually used to determine the length of crest vertical curves, whereas headlight sight distance, driver comfort, and appearance can be used to determine the length of sag vertical curves.

The normal procedure for designing a crest vertical curve is to provide a sufficient length of vertical curves to enable a driver to bring the vehicle to a stop after discerning an object 6 in high on the roadway ahead. The normal procedure for designing a sag vertical curve is to provide a sufficient length of vertical curve to enable a driver to bring the vehicle to a stop after the headlights illuminate an object on the roadway ahead.

The roadway of an LWSC may be wet and slick. In this case it is appropriate to use a lower friction factor in the stopping sight distance formula. Table 1 has been prepared based on a friction factor (f) equal to 0.20.

TABLE 1 STOPPING SIGHT DISTANCES FOR LOW WATER STREAM CROSSINGS

Velocity (mph)	Perception and Brake Reaction Distance (ft)	Braking Distance (ft)	Stopping Distance (ft)
5	18.4	8.3	27
10	36.8	33.3	70
15	55.1	75.0	130
20	73.5	133.3	210
25	91.8	208.3	300
30	110.3	300.0	410

Figure 5 has been prepared to enable the length of the crest vertical curves at an LWSC to be determined based on an eye height equal to 3.5 ft, an object height of 6 in, and a stopping sight distance from Table 1. Figure 6 has been prepared to enable the length of the sag vertical curves at an LWSC to be determined based on stopping sight distances from Table 1.

THE SELECTION OF MATERIALS

Each crossing has unique characteristics that are peculiar to the region in which it is located and the specific site under

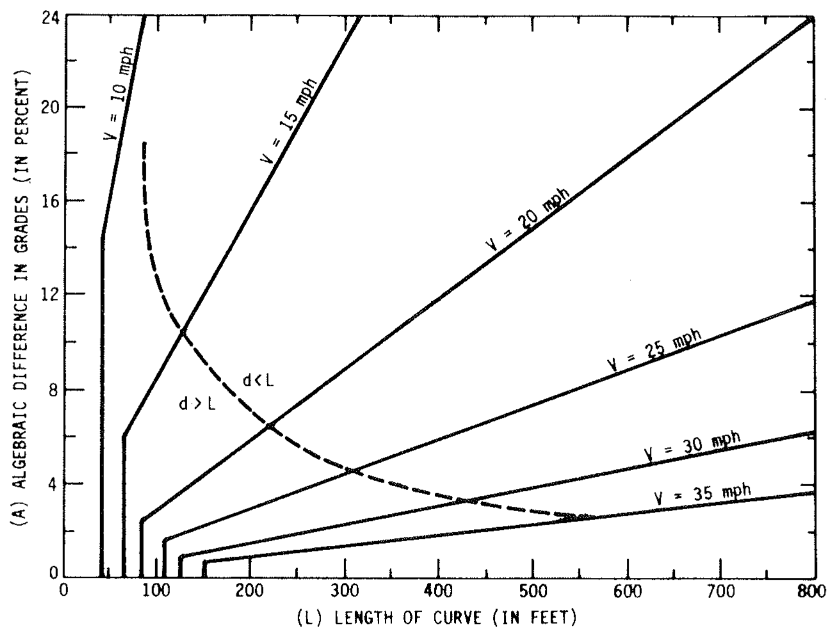

FIGURE 5 Minimum length of crest vertical curve for LWSCs (based on height of eye = 3.5 ft, height of object = 6 in, and stopping sight distance from Table 1).

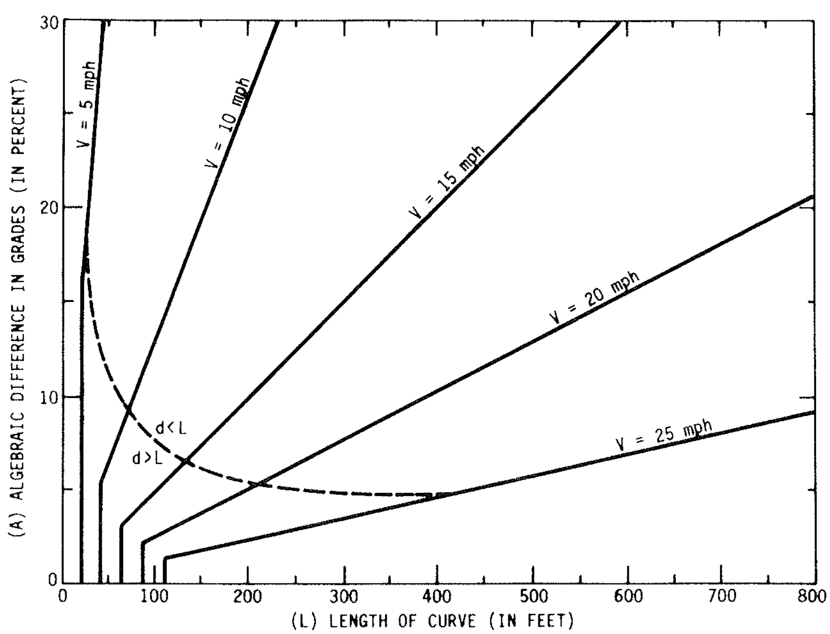

FIGURE 6 Minimum length of sag vertical curve for LWSCs (based on stopping sight distances from Table 1).

consideration. The stream gradient, channel geometry, soil characteristics, costs of materials and labor, and the relative importance of the crossing are all factors of concern.

Various materials can be used ranging from vegetation to Portland cement concrete. Low initial costs may require expensive maintenance. However, a low maintenance design may present overly high initial costs. Each site must be analyzed.

The following items of concern have been selected from studies by the New York Bureau of Soil Mechanics and Keown et al. as considerations in the selection of a suitable material to protect against erosion (*1, 2*).

- The forces that cause possible failure of the material, whether they be expressed in terms of velocity or tractive force, must be evaluated for each particular material. The specifications of the type or quality of suggested material will depend on the chosen design flood return period.
- The channel geometry in terms of bed slope and bank slope at a particular crossing location will need to be evaluated in order to calculate the forces acting on bank protection.
- Nonuniform settlement as a result of soft foundations and settlement as a result of scouring are important considerations in the design of nonflexible structures such as concrete or Fabriform.
- The environment may have an effect on the material; this includes the action of ice on riprap and sunlight on Fabriform.
- Economic considerations, such as the cost of materials, labor, and maintenance, are an important factor. Alternatives that have a low initial cost might require expensive maintenance, whereas low-maintenance structures might involve an overly high construction cost.
- Aesthetic considerations are considered to be largely unimportant because the structures will generally be located in relatively remote regions; however, this might be an important consideration in state parks.

Vegetative Protection

Two basic types of vegetation can be used as protective materials in crossings: grasses and woody plants. Woody plants take longer to establish than grasses but have the advantage of being more robust and having a greater retarding effect on the stream velocity. This means that woody plants are more suitable for higher velocities. Chow presented data produced by the U.S. Soil Conservation Service on the velocity resistance and retardance characteristics of woody plants (*3*). These data are given in Table 2. The maximum design velocity permitted for the use of grass is 5 ft/sec and is below that which most grasses are capable of resisting. The retardance effect is beneficial because it can reduce velocities close to the bank by up to 90 percent, thereby greatly reducing the eroding power of the flow. However, it has been found that those grasses with the largest degree of retardance also require the best growing conditions.

Environmental conditions for the use of grasses are very important. Steep sideslope angles can create conditions that facilitate erosion. Furthermore, grasses cannot be planted in locations where they will be subjected to anything other than short periodic flows.

A vegetative cover presents an aesthetically pleasing cross-section in a primitive environment at a low cost. Temporary initial protection by the use of a jute may be necessary. Vegetation is also easily maintained or replaced in the case of undermining or settlement.

TABLE 2 PERMISSIBLE VELOCITIES FOR VARIOUS TYPES OF VEGETATION

Cover (1)	Slope Range Percent (2)	Permissible Velocity, fps[1] Erosion-Resistant Soils (3)	Easily Eroded Soils (4)
Bermuda grass	0-5	8	6
	5-10	7	5
	>10	6	4
Buffalo grass, Kentucky bluegrass, smooth brome, blue grama	0-5	7	5
	5-10	6	4
	>10	5	3
Grass mixture	0-5	5	4
	5-10	4	3
	Do not use on slopes steeper than 10%		
Lespedeza sericea, weeping love grass, ischaemum (yellow bluestem), kudzu, alfalfa, crabgrass	0-5	3.5	2.5
	Do not use on slopes steeper than 5% except for side slopes in a combination channel		
Annuals—used on mild slopes or as temporary protection until permanent covers are established, common lespedeza, Sudan grass	0-5	3.5	2.5
	Use on slopes steeper than 5% is not recommended		

[1]The values apply to average, uniform stands of each type of cover. Use of velocities exceeding 5 fps only where good cover and proper maintenance can be obtained.

Rock Riprap

There are three basic types of riprap: dumped, hand-placed, and grouted. The dumped or hand-placed stones constitute a protective lining that is composed of multiple layers of stones that rest on the foundation soil or a bedding layer. The multiple layers ensure that the underlying soil is not exposed if settlement occurs or if scouring by ice or debris occurs.

In terms of cost, the best alternative is dumped riprap, which requires less labor cost. Grouted riprap is the most rigid material and the most susceptible to failure by undermining. Dumped riprap is the material that is least vulnerable to impact damage.

The size and grading of rocks to be used are important. A well-graded riprap acts as its own filter layer and prevents outwash of the underlying soil. A well-graded riprap can be thinner than a uniformly graded riprap with a special filter layer.

Soil Cement

Soil cement can be used as a substitute for riprap. This is especially useful where suitable stone is not available or is costly. Soil cement blocks can be cast at the site and hand-placed to guard against erosion. Soil cement is relatively inexpensive and portions can be replaced with ease. The labor of casting and hand-placing the blocks can be significant.

Soil, sand, and cement have been used to form an erosion-resistant surface. It must be placed under dry conditions and compacted. Shrinkage cracking and a low flexural strength may create problems.

Gabions

Gabions are wire baskets that are filled with stones. They have been used successfully on low water crossings. Reno mattresses and Fabriform are also examples of commercially available slope protection materials.

Gabions have the advantage of being flexible, which makes them less prone to settlement or undermining. They also fill up with silt and can support vegetative growth. Gabions are also usually cheaper than concrete. Suitable rock filler material must be available.

Reinforced Concrete

Reinforced concrete is the most elaborate and costly form of protection; it is also the most durable and requires the least maintenance costs. Designers must consider the use of suitable reinforcement to guard against undermining and scour.

Adjacent Erosion

When selecting a site, the designer should select a location where the stream is stable. If evidence of aggradation, degradation, or lateral migration is evident, an attempt should be made to relocate the crossing or provide remedial measures.

If the designer determines that erosion adjacent to the crossing may occur, erosion-resistant materials or cut-off walls should be provided. The exit velocity, depth of scour, and length of stilling basin must be estimated.

Seepage Considerations

Two potential problems can arise as a result of subsurface seepage beneath hydraulic structures: excessive uplift pressures and piping. The probability of these problems increases with an increasing head difference between the upstream and downstream sides of the crossing. The difference in head may not be large in vented fords, whereas head differences of more than 2 ft might occur in a case in which a ford is used. A flow net analysis was performed using typical ford geometries and sediment properties for a 2-ft head difference. This analysis indicated that, without any cut-off for seepage control, it is unlikely that problems of excessive uplift pressures and high exit gradients will occur; cut-offs for seepage control would therefore be unnecessary. However, if the designer anticipates unusual conditions, a flow net analysis should be conducted to evaluate both pore pressure distribution and exit gradients for conditions of no cut-off and various cut-off geometries.

Although a cut-off may not be justifiable as a means of seepage control, it may be necessary as a protection against scour. The presence of a cut-off wall on the downstream side of a low water crossing will have the effect of decreasing seepage quantities and decreasing exit gradients relative to a condition of no cut-off. However, the cut-off will have a tendency to increase uplift pressures on the downstream side of the crossing. Therefore, it is recommended that if a cut-off is designed for scour control, the structure should be analyzed with a flow net to ensure that the pore pressures are not excessive.

CONSTRUCTION DETAILS

A detailed construction procedure is not practical because of the wide range in the variables of materials and site characteristics. However, certain elements of construction have been successfully used and are included here as examples.

The various components of an LWSC are shown in Figure 2. The design elements were described earlier. The use of cables to hold pipes in place in case the core material is washed out is shown in Figure 7. Examples of side walls and cut-off walls are depicted in Figure 8. These devices are used to protect the edges of the crossing and to prevent erosion of the core filler material. An example of erosion protection for a high type of crossing is shown in Figure 9. The extent to which crossing material can be provided is depicted in Figure 10. Different types of unvented ford protection are shown in Figure 11.

TRAFFIC CONTROL

A low water stream crossing has two unique characteristics that are not associated with a traditional bridge. The vertical profile at the crossing is usually restricted to low speeds and the pavement surface is subject to periodic flooding. Adequate warning of these conditions should be provided to the user. The following recommendations are based on recent research by Carstens and Woo (*4*).

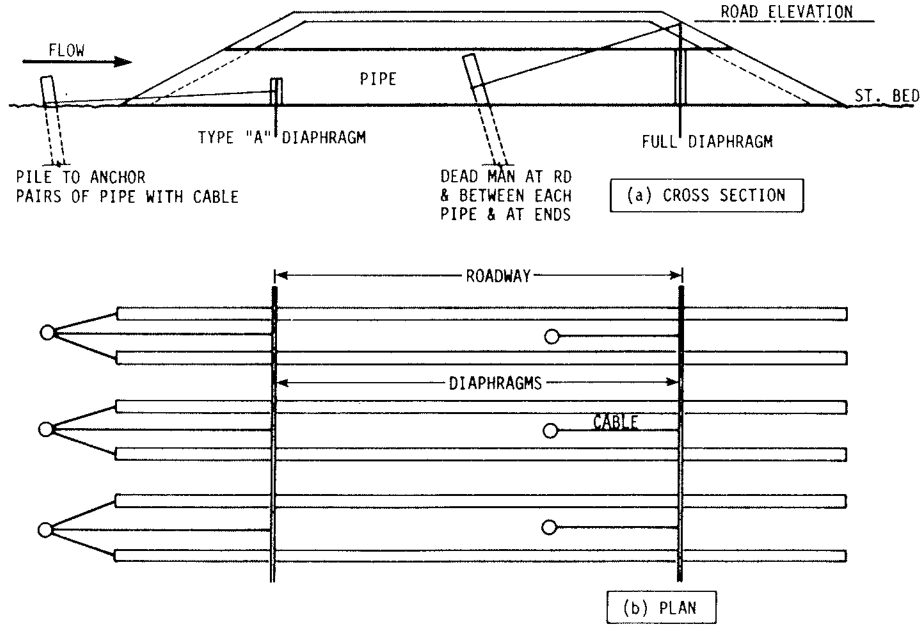

FIGURE 7 Cable anchor details.

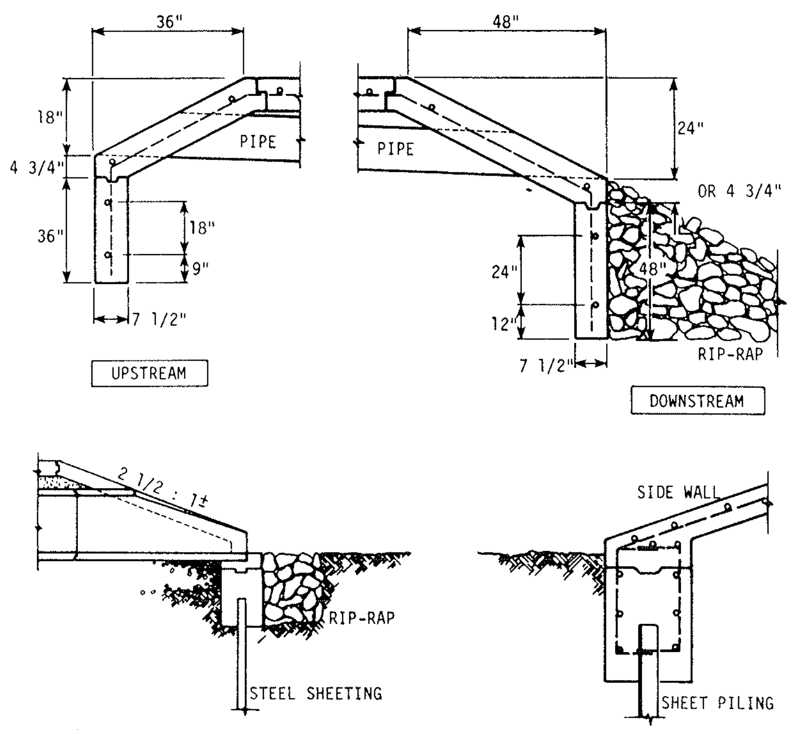

FIGURE 8 Typical sidewall and cut-off wall sections.

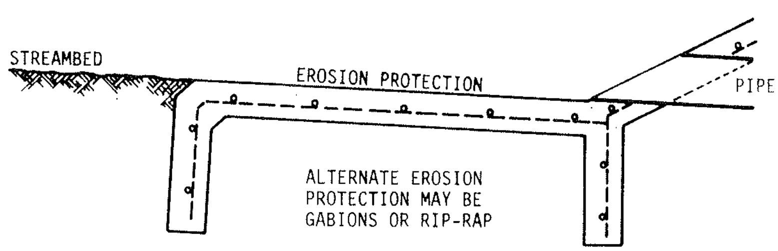

FIGURE 9 Typical erosion protection.

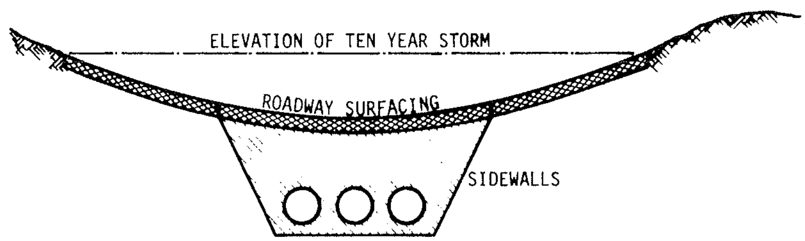

FIGURE 10 Minimum limits of LWSC roadway surfacing.

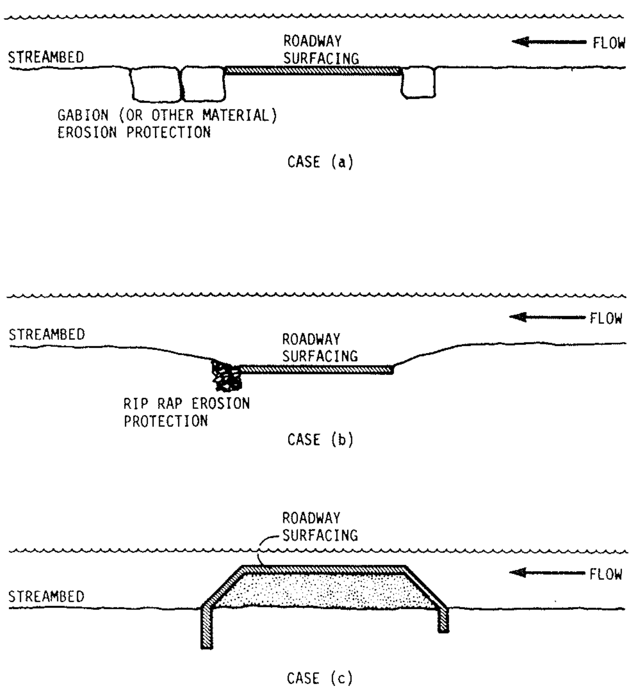

FIGURE 11 Typical fords—roadway cross section.

Application of a Low Water Stream Crossing

In a survey of LWSC use in the United States, 61 percent of the respondents reported they were used only on unpaved roads (*4*). Because paved highways have a geometric design and traffic control that are conducive to higher speeds, drivers' expectations are not consistent with the vertical profile encountered at LWSCs. In addition, because unpaved roads are limited to low traffic volumes, the use of LWSCs on these roads would involve a lower exposure to traffic. Carstens and Woo do not recommend the use of LWSCs on paved roads in Iowa.

The use of an LWSC design is based on an acceptance of periodic flooding. If flooding would isolate a place of human habitation, either an alternate design should be considered or an alternate emergency access route should be developed.

Approach Signing

The signing recommendations shown in Figure 12 are based on Carstens' and Woo's research (*4*). The recommendations were subsequently adopted by the Iowa DOT as recommended practice. According to Carstens and Woo, the intent of the regulatory sign DO NOT ENTER WHEN FLOODED is to preclude travel across the LWSC when the roadway is covered with water (*4*). Such a regulatory sign requires a resolution by the Board of Supervisors. The adoption of this sign in effect significantly reduces the applicability of an unvented ford.

Supplemental Signing

If the location of an LWSC is not apparent from a point approximately 1,000 ft in advance of the crossing, a supplementary distance plate may be used. The message "700 feet" would be displayed with the FLOOD AREA AHEAD sign. The sign would be 24 in × 18 in with a black legend on a yellow background.

An advisory speed plate may be used if the maximum recommended speed at the LWSC is less than the speed limit in effect, which is usually the case. The advisory speed plate would be installed in conjunction with the FLOOD AREA AHEAD sign. However, if a supplemental distance plate is used, the advisory speed plate would be installed in conjunction with the IMPASSABLE DURING HIGH WATER sign.

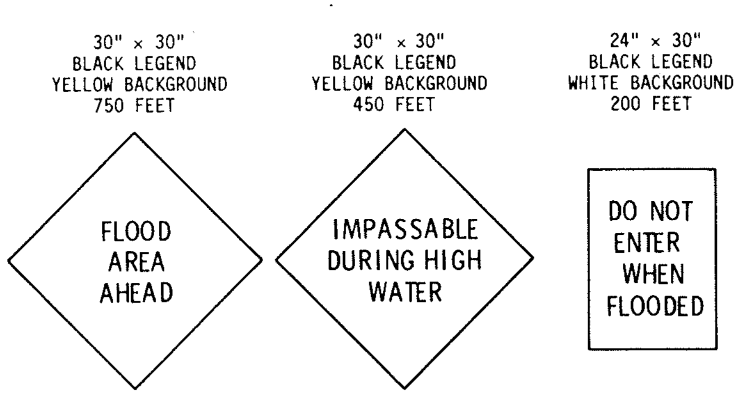

FIGURE 12 Signs recommended for installation at low water stream crossings.

Controls at a Low Water Stream Crossing

Various controls have been used to delineate the edges of the traveled way at an LWSC. Curbs are generally unacceptable because the flow of water tends to deposit mud and debris on the roadway. Attempts have been made at a few locations to create a series of small, raised curb blocks with tapered upstream slopes to provide for a smooth laminar flow. The use of any projections above the normal roadway surface will have an adverse effect on the self-cleaning aspect of the smooth cross-section. However, observations of existing applications, or further research in this area, are needed.

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Guidelines for the Design of Low-Cost Water Crossings

LOUIS BERGER, JACOB GREENSTEIN, AND JULIO ARRIETA

In Ecuador, as in many Third World countries, low-volume rural roads can only be economically justified when very low-cost bridges and simple water crossings (fords) are used. Traffic analyses indicate that in most cases the trucks that travel these roads carry loads that weigh less than 6 to 10 metric tons. Therefore, most of the drainage structures are designed to carry only 10 tons on two-axle light vehicles. Roads are designed according to AASHTO HS-15 standard loading in those locations where heavy traffic is generated from timber production or banana plantations. The standard AASHTO HS-20 live load cannot be economically justified for these low-volume roads. The traffic volume in rural regions is very low, which enables such economical structures as graveled fords to be used, and, when economically feasible, one-lane bridges with either complete or split decks. The relationships between the type of material, the span or length of the superstructure, and the cost are analyzed. It is primarily concluded that simple timber bridges made of stringers and transverse laminated decks are the most economical solutions for simple spans up to 17, 14, and 10 m for 6-, 10-, and 24.5-ton truckloads, respectively. Simple-span, split-deck, reinforced-concrete superstructures are feasible for spans of up to 30 m. Spans can be as long as 45 m if prestressed girders are used. Suspension bridges with timber decks and timber-stiffening trusses were built to carry 6-ton trucks or cattle wagons and were more cost-effective than timber or concrete structures. It was concluded that with the judicious reduction of the design standards of live loads, cross-sections, geometry, material specifications, and hydrologic and hydraulic considerations, construction costs could be reduced by 50 percent or more. These savings make it possible to justify the construction of many low-volume rural roads that would otherwise be impossible to finance.

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Low-volume roads are needed in such developing countries as Ecuador and Colombia to provide access in agricultural and rural regions (*1, 2*). A socioeconomic analysis is performed to determine which type of road is the most economical to build. The use of this methodology enables the least-cost road to be determined for any given traffic projection, degree of agricultural productivity, and extent and type of social and population activities.

Several types of low-cost rural roads exist in Ecuador: (a) earth or dirt roads that are 2.5 to 4.0 m wide and provide access only during the dry season, (b) 4.0- to 6.0-m-wide compacted subgrade or gravel roads, (c) 4.0- to 5.0-m-wide stone roads constructed mainly in the Andes region, and (d) 6.0- to 7.2-m-wide base course roads with or without blacktop. Construction of most of these low-volume roads can be economically justified only if the construction cost is minimized to achieve a feasible rate of return on the investment. The minimum initial rate of return required to justify investment in the construction of low-volume roads in Ecuador in 1984 to 1985 was 12 percent. This objective can be achieved only if low-cost water crossings are used to provide access.

The economic horizon or lifetime of a low-cost road in Ecuador is 17 years. It was concluded in a study financed by the World Bank that all agricultural and other economic benefits could be achieved during this 17-year period, and the investment therefore would be justified (*2*). The minimization of costs and maximization of benefits during the economic horizon are both needed to optimize and justify road and bridge construction. Cost savings can be obtained by setting appropriate standards for certain design elements, such as design load, cross-sections, low-cost materials, and hydrologic and hydraulic design criteria, even though these criteria may appear to be substandard to the developed world.

Typical low-cost bridges and water crossings in Ecuador are shown in Figures 1 to 5. A one-lane timber bridge in Puerto Viejo, Ecuador, that was designed to carry only one vehicle at a time with a total weight of less than 6 tons is shown in Figures 1a and 1b. A one-lane timber deck in Puerto Bartelo, Ecuador, that was designed to support a truck carrying less than 10 tons is shown in Figures 2a and 2b. A typical one-lane concrete and steel split deck that can carry one truckload of less than 10 tons is shown in Figures 3a and 3b. A one-lane concrete bridge that was designed to carry HS-15-44 trucks with a total weight of 24.5 tons is shown in Figures 4a and 4b. A ford-type water crossing is shown in Figures 5a and 5b. This type of ford is very common in the Ecuadoran Andes.

Basic design guidelines, typical cross-sections, and cost comparisons of low-cost bridges and water crossings in Ecuador are presented.

LIVE LOAD DESIGN

It is well-known that the transport of goods in the Third World is mainly performed by the private sector, which saves money by overloading its trucks. Recent projects in Ecuador and Colombia financed by the World Bank indicate that little is currently being done on the main roads to control truck overloading. This conclusion also appears to apply to other countries. As a result of this evidence, the structural division in the Ecuadoran road authority designed all bridges for both main and rural roads according to the AASHTO HS-20 standard truck loading. The lower AASHTO standard HS-15

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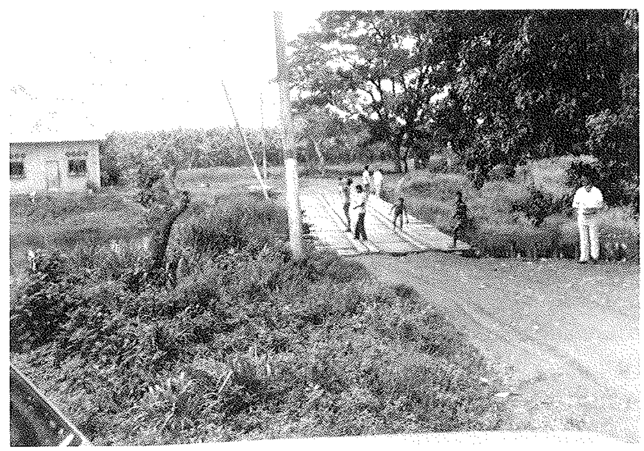


(b)

FIGURE 1 Two views of a timber bridge.

(a)

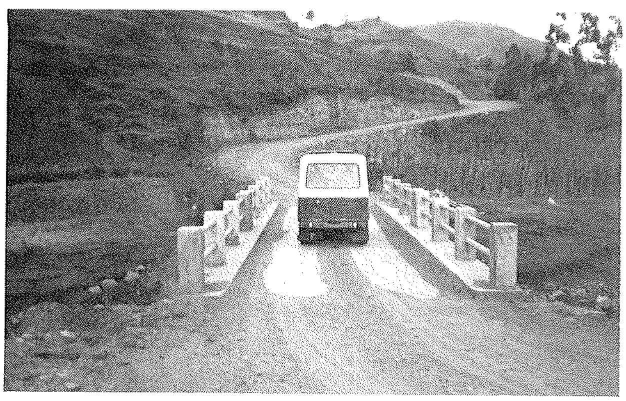
(b)

FIGURE 2 Two views of a timber deck.

(a)

(b)

FIGURE 3 Two views of a one-lane, split-beam deck.



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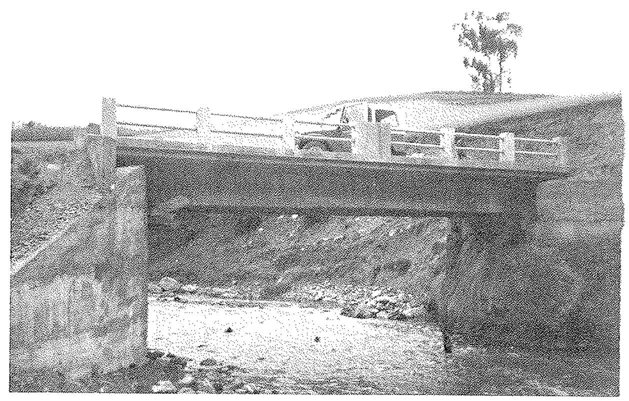

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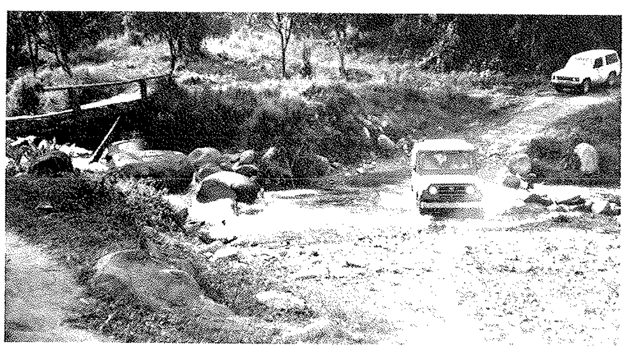
FIGURE 4 Two views of a one-lane concrete bridge design for HS 15-44.



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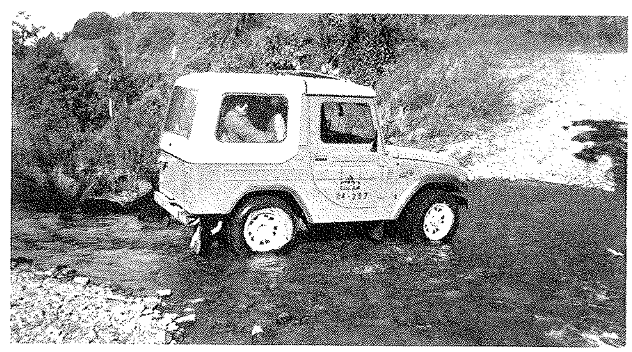


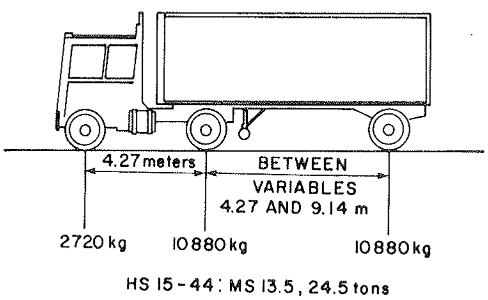
(b)

FIGURE 5 Two views of a low-cost ford in the Ecuadoran mountainous zone.

truck was only occasionally used in the design of rural bridges. Recent economic and transport studies in Ecuador indicate that the actual vehicle loading on rural roads is significantly lower (*1-3*). The largest vehicle on over 90 percent of the roads is a two-axle truck with a total weight of less than 10 metric tons. About 75 percent of the vehicles are pick-up trucks and light buses and trucks, with a total weight of 2 to 6 tons. An economic and traffic projection analysis indicated that the volume of traffic might increase slightly in most of the existing Ecuadoran rural roads, but no changes in vehicle type or total weight are expected (*1, 2*). In other words, the projected demand and economic growth, and the low standard of the road and pavement, make the use of oversized or overloaded vehicles infeasible (*1, 4*). Only in a few rural locations—regions with heavy traffic from timber-producing regions or banana plantations—can a AASHTO standard HS-15 live load be economically justified. These relatively few roads usually have higher design standards; a 6.0- to 7.2-m-wide base course pavement with or without blacktop is usually used. Based on these economic and traffic forecast analyses, the Ecuadoran road authorities decided that it was practical and economical to adopt lower design standards for live loads on most of the low-cost rural bridges. The following three load categories were adopted (see Figure 6):

- An M6 truck with 1,200 kg and 4,800 kg on the front and rear axles, respectively;
- An M10 truck with 2,000 kg and 8,000 kg on the front and rear axles, respectively; and
- An HS-15 load with 2,720 kg on the front axle and 10,880 kg on each of the two rear axles, for a total of 24,480 kg.

The AASHTO standard HS-20 live load was not found to be economically justified for these low-cost roads.



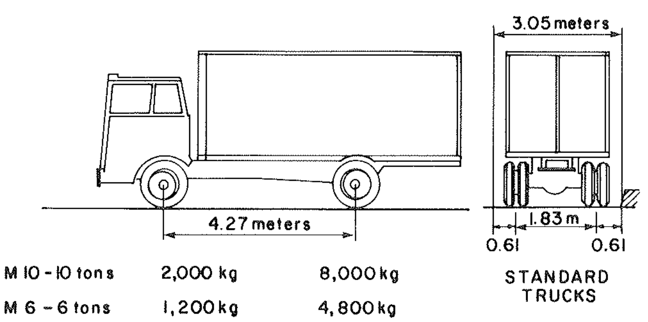


FIGURE 6 Two designs for live loads on rural bridges.

HYDRAULICS AND HYDROLOGY

The deforestation that occurred in Ecuador the last few years has resulted in severe flooding in rural regions. The deforestation also caused an increase in the flooding discharge and a reduction in its duration. The deforestation and changes in flooding characteristics occurred more often in the rural or remote regions and less often in the vicinity and area of influence of major highways. Previous hydraulic records for rural bridges therefore should be analyzed with caution to prevent rural bridges from failing during floods, as shown in Figures 7a and 7b. A typical scour failure in a rural road bridge in the Ecuadoran province of El Oro is shown in Figure 7a. The scouring caused 1 ft of settlement in the center abutment. The bridge is still used for light and partially loaded trucks.

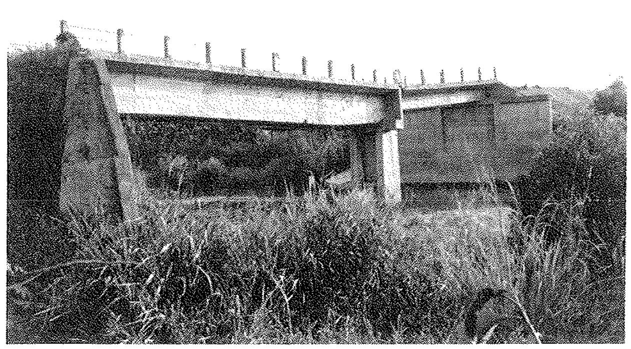
A typical total bridge failure that was caused by flooding in 1982 is shown in Figure 7b. Although the hydraulic analysis should be precisely executed to eliminate any unexpected failures, especially in cases in which changes have recently occurred in the flooding pattern, the hydraulic design criteria should permit bridge construction costs to be minimized. The criteria for main roads specify or require that the clearance between the bridge's bottom deck and the maximum flood level should be 1 ft (30 cm) for the occurrence of a storm every 100 years. In rural road design, the storm period can be reduced to 25 years. This period is approximately equal to 150 percent of a road's economic lifetime. Experience also indicates that the water clearance should remain at 1 ft unless the water velocity is slow and accumulation of debris is not expected. A slow stream is defined as one in which the slope is less than 0.5 percent and the maximum velocity is below 10 ft/sec.

LOW-COST BRIDGES

Concrete Bridges

Before 1984, the Ecuadoran rural bridges were designed according to AASHTO standards (*5*). A cross-section of a typical two-lane bridge is shown in Figure 8. Such bridges were designed in 1980 to carry an AASHTO standard HS-20 truck. Because two-lane bridges were found to be economically infeasible for rural roads, a new standard was established (*1, 2*).

A one-lane bridge (Figures 9a and 9b) was established as the highest standard that could be economically justified for rural



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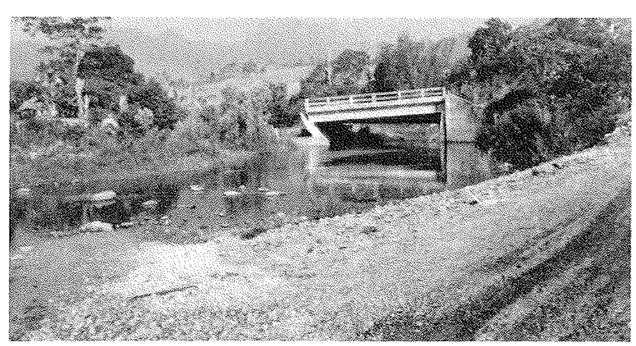
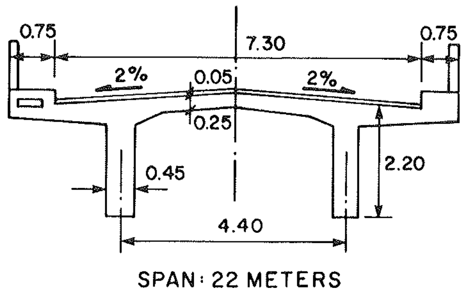
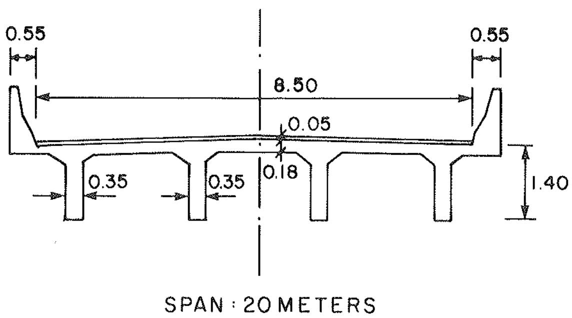


(b)

FIGURE 7 Two views showing typical bridge flood failure in the province of El Oro.

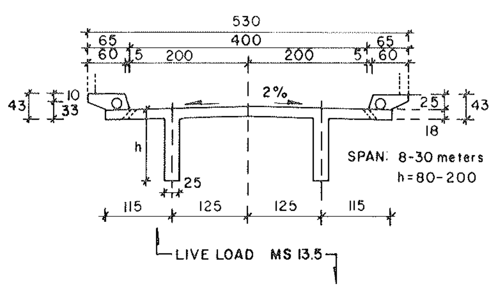


(a) ESTERO LUNA GRANDE, ECUADOR

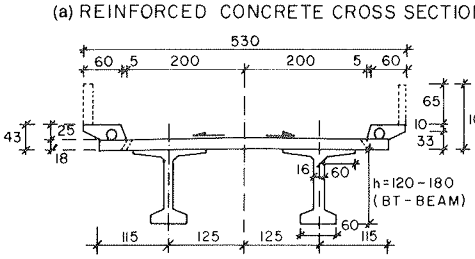


(b) ESTERO LUNA CHICO, ECUADOR

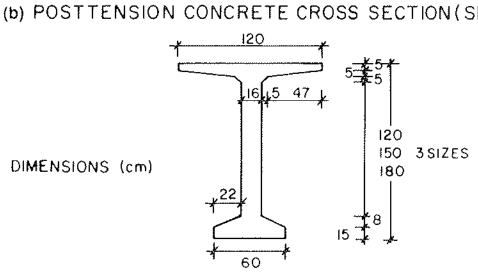
FIGURE 8 Typical two-lane rural bridge design for AASHTO HS-20 truck loading in Ecuador.



(a) REINFORCED CONCRETE CROSS SECTION



(b) POSTTENSION CONCRETE CROSS SECTION (SPAN 24 TO 42 M)



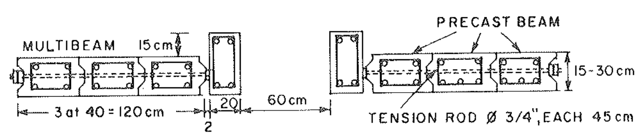
(c) BULB-T BEAM

FIGURE 9 (a) One-lane reinforced concrete bridge cross-section; (b) one-lane post-tensioned concrete cross-section.

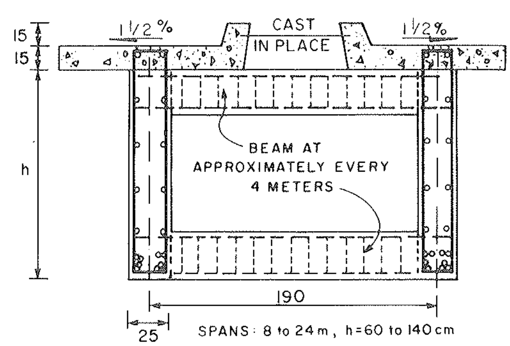
roads. The bridge cross-sections shown in Figures 9a and 9b are designed to carry an AASHTO standard HS-15 truck. A reinforced concrete cross-section that is usually feasible for a span of 8 to 30 m is shown in Figure 9a. A post-tensioned concrete cross-section that is usually feasible for spans between 25 and 45 m in length is shown in Figure 9b. The sidewalk and guardrail shown in Figure 9b are provided when pedestrians, cattle, and vehicles are to use the bridge. Cattle can use the bridge only if it crosses deep water.

An economical one-lane, split-deck bridge is shown in Figures 10a, 10b, and 10c. The split cross-sections shown in these figures are used for 6- and 10-t trucks (M6 and M10). A simple, multibeam, precast concrete bridge is used for short spans of usually 8 m or less. The construction procedure is very simple. The slabs are precast on the river bank and are easy to place and tie together to form the split deck. A typical cast-in-place split-deck bridge that is primarily used for bridge spans of 8 to 24 m is shown in Figure 10b. Cast-in-place, reinforced concrete bridges are popular in developing countries, such as Ecuador, Peru, and Colombia. This technique is also well-known to local industry. In addition, the only materials that have to be transported to the site are the reinforcing steel and cement.

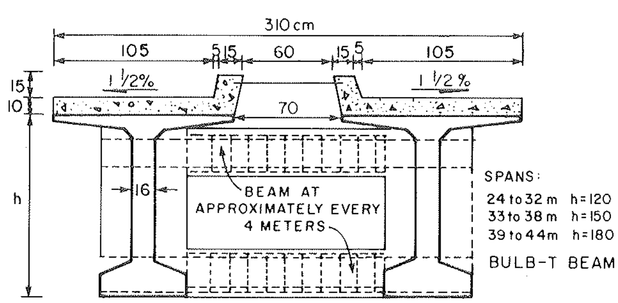
The simple, split-lane reinforced bridge shown in Figure 10b is light and safe to use by vehicles and pedestrians. It is worthwhile to mention that the geometry of the split lane with the inside curb shown in Figures 10a and 10b contributes to its traveling safety. The split lane contributes naturally to speed



(a) MULTIBEAM SPLIT DECK



(b) PRECAST DECK



(c) POSTTENSION CONCRETE BEAM

FIGURE 10 Typical split deck for low-cost bridges.

reduction and the inside curb is effective in preventing vehicles from sliding off the bridge. A split deck with prestressed girders of the sort shown in Figure 10c is more feasible in Ecuador when the single span exceeds 24 m.

In developing countries, such as Ecuador, Colombia, and Peru, the required pretensioning equipment is usually not available at a reasonable distance from the site; therefore, most of the prestressed bridge elements are post-tensioned. Standard post-tensioned girders and cast-in-place slabs that are practical for low-cost bridges with spans ranging between 25 and 45 m are shown in Figure 10c. Long, low-cost bridges are rarely constructed with a single span of over 45 m.

Each of these cases has been studied in detail in regard to the live load and bridge element type. The live load of bridges on roads classified as low-volume and low-cost in Ecuador consists of cattle or a maximum truckload of 6 tons.

Timber Bridges

In the past few years timber has played an important economic role in the construction of low-cost bridges in countries that comprise the Pacto Andino, which include Colombia, Venezuela, Ecuador, Peru, and Bolivia. These countries are rich in natural resources, especially in their huge tropical and subtropical zones. New technology in regard to timber structural elements is provided through the Agreement of Cartagena-Colombia. This agreement provides assistance and technology for the classification of timber, improved mechanical properties, pest control, and processing and treatment of timber (*6*). Further information on the design and use of timber bridges is available elsewhere (*7-9*).

It was determined that the currently unlimited source of natural raw materials, and the availability of technical assistance in production, make timber a feasible and practical alternative for bridge construction (*6*). Two other factors contribute to the feasibility of using timber for rural bridge construction. First, timber elements, especially in decks, are light; they weigh approximately one-third to one-fourth of the weight of an equivalent concrete deck. The use of timber therefore reduces foundation costs. Second, the economic life of a rural road is only 15 to 20 yrs; therefore, the investment in lumber treatment can and should be limited to this period only.

A simple Ecuadoran timber bridge with a laminated deck is shown in Figures 11a and 11b (*3*). The cross-sections are made of timber stringers that span between abutments or piers and transverse laminates that are nailed to one another and to the stringers. The transverse laminates are sometimes spanned over trussed or solid-web girders. The timber deck cross-section shown in Figure 11b was designed to a maximum span length of approximately 17, 14, and 10.5 m for truckloads of 6, 10, and 24.5 tons, respectively. This structure is economical, quick and easy to construct, and easy to maintain.

The design guidelines for the dimensions and optimum location or separation of the stringers to carry these traffic loads are shown in Figure 11b (*3*). For example, for a span of 10-m, relatively heavy stringers 25 cm wide and 50 cm high should each be spaced at 1.35, 0.85, and 0.60 m to carry live loads of 6, 10, and 24.5 tons, respectively. The maximum and most economical timber deck span can be increased by approximately 40 percent by using longitudinal or transverse cable post-tensioning. The implementation of this technique is still very limited in the developing countries of South America.

Another type of low-cost timber bridge was developed and implemented by the United Nations Organization for Industrial Development (UNOID). This prefabricated timber bridge consists of triangular moduli 3 m long that are joined together at the site. These short elements are easy to transport. Like the other timber bridges that were previously described, the UNOID bridge can be assembled easily by unskilled labor.

A typical cross-section of a UNOID bridge is shown in Figure 12. This one-lane timber bridge is now promoted in

(a)

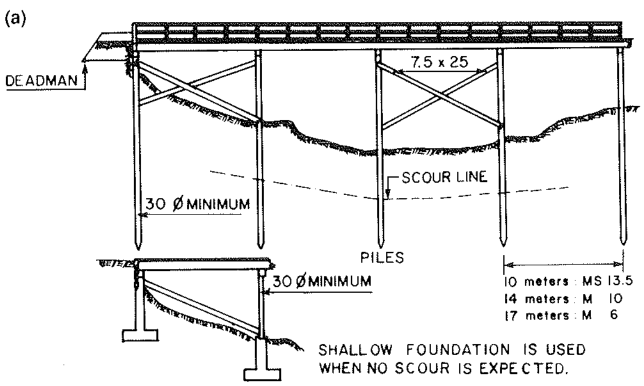


FIGURE 11 Typical timber bridge with laminated deck.

(b)

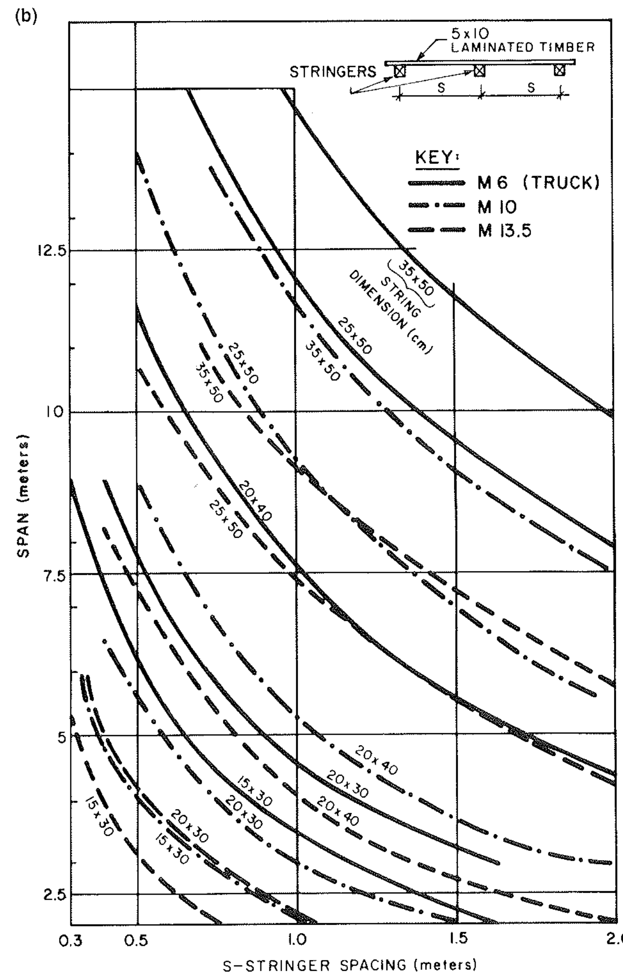


FIGURE 11 continued

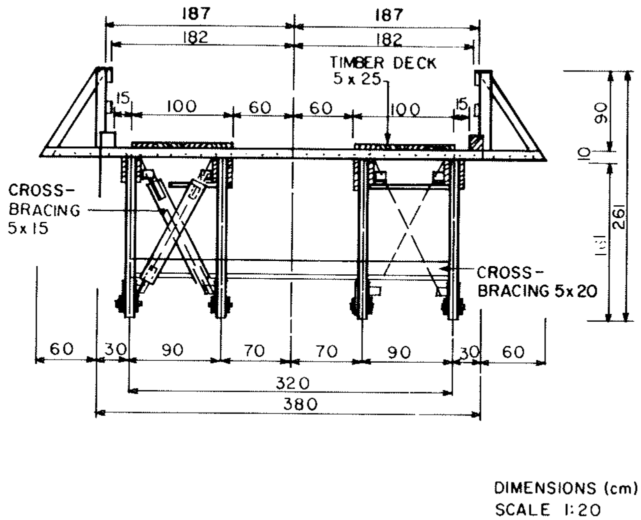


FIGURE 12 ONOID timber bridge.

Africa, Central America, and Ecuador by the United Nations and is designed to carry 6-, 10-, and 24.5-t trucks in a single span of 24, 21, and 15 m, respectively. As was mentioned previously, a long, single-span low-cost bridge is only occasionally needed for both vehicles and cattle. This special need primarily exists in the eastern tropical Ecuadoran Amazonas region. A typical bridge is shown in Figure 13. This bridge is designed only for light vehicles of 6 t or less, or for passing cattle. The spans of this single bridge vary between 45 and 120 m; the only nontimber elements are the reinforced concrete towers, the anchor blocks, and the cable.

Other Low-Cost Water Crossings

In many cases in which international or government funds are limited and timber for split-deck bridges is unavailable, other

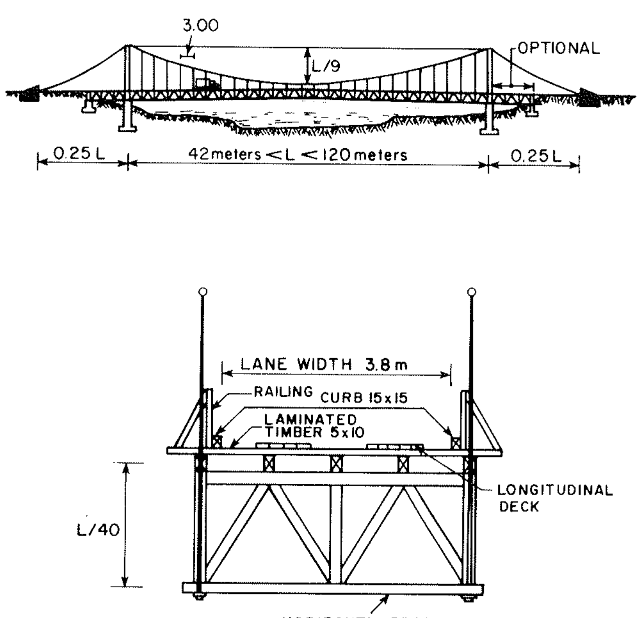


FIGURE 13 Single-span bridge with timber deck and timber-stiffening trusses.

low-cost water crossings can be built. In most of these cases, good judgment and experience with locally available materials can be used instead of standard specifications and structural analysis. The most commonly used and economical types of water crossings in Ecuadoran rural regions are described in the following paragraphs. All three types provide reasonable access for 3 to 5 yrs with no major repairs.

One type of low water crossing consists of beams of solid webs that are nailed to two flanges made of two layers of boards that support a split in the timber deck. This structure is used in spans of up to 25 m and mainly carries vehicle loads of less than 6 tons. This structure can deteriorate rapidly if moisture accumulates between the boards.

A no-stringers deck is designed for span lengths of up to 12 m. The bridge is made of timber laminates that run parallel to its longitudinal axis. These laminates are the only element in the superstructure.

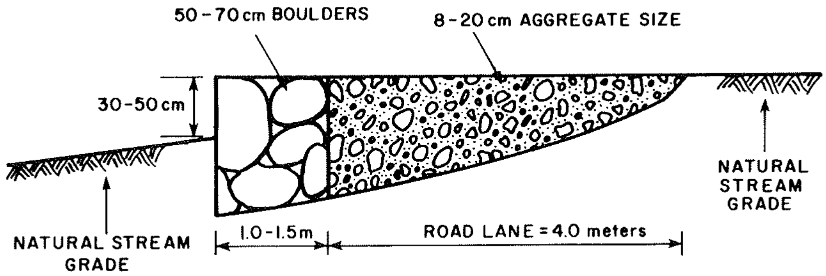
Graveled fords are commonly used in the mountainous regions of Ecuador (Figure 5). Fords are used as low-cost water crossings on almost every unpaved rural road in this region. A typical cross-section of a ford is shown in Figure 14 for both steep and flat water crossings. The construction of this type of crossing is usually labor-intensive. The surface of graveled forms usually performs adequately for 3 to 5 yrs in the Ecuadoran Andes. Maintenance is rather simple; it is also performed by manual labor with a relatively minor cost. Experience in Ecuador clearly indicates that the construction and maintenance costs of a ford are always less than a fraction of those for a single-lane, low-cost bridge. They cost approximately 50 to 100 U.S. dollars/linear meter when local materials are available.

COST COMPARISON

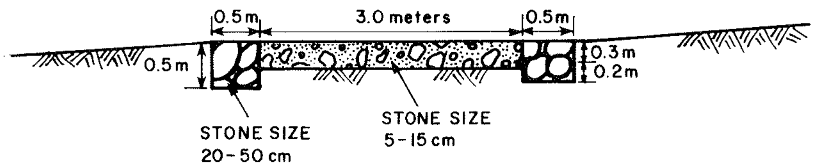
The construction costs of rural concrete bridges that are designed to carry AASHTO standard HS-20 trucks are given in Table 1, in which the costs of the bridge deck and the bridge as a whole are broken down. As shown in Table 1, the total construction cost of a two-lane, 8.5-m-wide concrete bridge varies between 1,600 and 2,200 U.S. dollars/linear meter (1986 prices). The cross-section of this typical bridge is shown in Figure 8.

Significant savings in cost were achieved in Ecuador by using the low-cost bridges described in this paper. The savings in cost are given in Table 2, in which the relationships between the total bridge construction cost per linear meter, traffic loading, and type of bridge are shown. It is clearly indicated in Table 2 that the use of timber bridges significantly reduces the cost of construction. The average total cost of a one-lane timber bridge in Ecuador in 1984 and 1985 was approximately 400, 500, and 650 U.S. dollars/linear meter (1986 prices) for truckloads of 6, 10, and 24.5 tons (MS 13.5), respectively. This cost is approximately 20 to 30 percent of that of the two-lane, standard concrete bridge that was previously used in Ecuadoran rural regions (Figure 8b).

The concrete split-deck bridge is another economical water crossing. The average total construction cost of the multibeam bridge shown in Figure 10a varies between 450 and 750 U.S. dollars/linear meter, which is approximately 30 to 45 percent of that of a two-lane standard bridge. The higher cost values are related to heavier truck loads. This bridge type is recommended



(a) STEEP SLOPE WATER CROSSING



(b) FLAT SLOPE WATER CROSSING

FIGURE 14 Simple ford cross-section (Ecuadoran mountainous zone).

TABLE 1 CONSTRUCTION COSTS OF RURAL BRIDGES IN ECUADOR IN 1984

Bridge width	Element	Economic cost x 10^3 (U.S. $/linear meter)
6.0 meters	Deck only	0.6 - 0.7
	Entire bridge	1.2 - 1.6
8.5 meters	Deck only	0.8 - 1.0
(Fig. 8.b)	Entire bridge	1.6 - 2.2

for short spans of about 8 m. The construction cost of the concrete split-deck bridge shown in Figures 10b and 10c usually varies between 400 and 900 U.S. dollars/linear meter for a truckload of 10 tons. This cost is approximately 28 to 40 percent of that of a standard two-lane bridge. The prestressed split-deck bridge is more economical when the single span of the bridge is over about 30 to 45 m.

Additional cost savings can be obtained by constructing a ford, as shown in Figures 5 and 14. The average construction cost of a one-lane ford was 50 to 120 U.S. dollars/linear meter. The variation in cost reflects the availability and cost of local materials and skilled labor in the vicinity of the project.

SUMMARY AND CONCLUSIONS

Investment in low-volume roads in developing countries can be economically justified only when very low-cost bridges and simple water crossings are used. Cost savings can be obtained by setting appropriate standards for the design elements of load, cross-sections, low-cost materials, and hydrologic and hydraulic design criteria, even though these criteria may appear to be substandard in the developed world. The following conclusions can be made.

Economic projection and traffic analysis indicate that truckloads of less than 6 to 10 metric tons are usually traveling along these roads. Therefore, most of the drainage structures are designed to carry 6 to 10 tons on two-axle, light vehicles. The AASHTO HS-15 or MS 13.5 loadings are used in the design of these bridges only when heavy traffic is expected from timber-producing regions or banana plantations. The AASHTO standard HS-20 live load cannot be economically justified for these low-volume roads.

The economic lifetime of a rural road in Ecuador was determined in 1984 and 1985 to be 17 years. All agricultural and other benefits can be achieved during this 17-year period. The investment in construction and maintenance costs can therefore be justified.

The traffic volume in Ecuadoran rural regions is often less than 100 vehicles per day; in fact, it is usually less than 20 to 50 vehicles per day. These low traffic volumes enable the use of graveled fords, such as those shown in Figures 5 and 14. The average construction cost of a one-lane ford was 50 to 120 U.S. dollars/linear meter. These timber bridges were found to be the most practical and economically justifiable bridges for simple spans of up to 17, 14, and 10 m, with 6-, 10-, and 24.5-ton truckloads, respectively.

The recommended storm period in the design of bridges on rural roads is 25 years. The recommended clearance between the maximum storm water level and the bridge should be 1 ft, unless the water velocity is very low. In cases in which the stream's ground slope is less than 0.5 percent, the maximum velocity is less than 10 ft/sec, and no accumulation of debris is expected, the water clearance could be reduced from 1.0 ft to 0.5 ft.

TABLE 2 COST COMPARISON OF LOW-COST BRIDGES (U.S. $1,000/linear meter)

	Bridge length (meters)				
Bridge type and loading	8 - 10	15	20 - 21	30	39 - 40
Timber bridge (Fig. 11)					
Load					
M6	.35-.45	.35-.45	.30-.40	.40-.55	.45-.60
M10	.40-.51	.43-.59	.45-.65	.45-.65	.54-.75
MS 13.5 (HS - 15)	.45-.62	.56-.77	.56-.77	.56-.80	.65-.90
Timber bridge (UNOID; Fig. 12)					
Load: MS 13.5	.62-.86	.46-.64	.43-.59	.48-.66	.46-.64
Multibeam (Fig. 10.a)					
Load					
M6	.40-.55	–	–	–	–
M10	.50-.68	–	–	–	–
M13.5	.62-.86	–	–	–	–
One - lane reinforced precast concrete split deck (Fig. 10b)					
Load: M10	.40-.60	.40-.60	.48-.66	.53-.73	.70-.97
Full-width deck (Fig.9a)					
Load					
M10	.50-.68	.54-.75	.61-.84	.67-.92	.88-1.21
MS 13.5 (HS - 15)	.63-.86	.72-.99	.77-1.10	.85-1.17	.93-1.28
One - lane prestressed split deck (Fig. 10c)					
Load : M10	–	–	–	.60-.84	.60-.88
Full - width deck (Fig. 9b)					
Load : M13.5 (HS - 15)	–	–	–	.80-1.10	1.04-1.43

Timber bridges made of stringers and laminated decks (Figure 11) appear to be an economical and practical solution for low-cost water crossings. The total construction cost per linear meter of one-lane timber bridges in Ecuador in 1984 and 1985 was as follows:

Load	*$ U.S.*
6-ton truckloads (M6)	350 to 450
10-ton truckloads (M10)	400 to 650
24.5-ton truckloads (MS13.5)	450 to 750

These costs are in the range of 20 to 40 percent of an AASHTO standard, two-lane bridge design that was previously used in the rural regions.

One-lane, split-deck, reinforced concrete bridges (Figures 9 and 10) obviously have a longer life expectancy than timber bridges and they can still be considered as adequate, economical alternatives. The most practical alternatives are multibeam, reinforced, and prestressed beam decks, as shown in Figures 10a, 10b, and 10c, respectively. The average total costs of these bridges are about 450 to 800 U.S. dollars/linear meter. The split-deck, reinforced concrete simple span has been found to be the most practical and economically justifiable bridge type for simple spans of up to 30 m. Spans can reach 45 m when prestressed girders are used.

The total construction cost of a full-width, one-lane, precast concrete bridge (Figure 9a) is approximately 20 to 30 percent more expensive than the equivalent split-deck bridge shown in Figure 10. It should be noted that the full one-lane prestressed bridge is always designed to carry HS-15 truckloads, whereas the split-deck bridge is designed for a standard 10-ton truck (M10).

It can be concluded that with the judicious reduction of design standards of live loads, cross-sections, geometry, material specifications, and hydrologic and hydraulic considerations, the construction costs of water crossings can be significantly reduced. This also means that it is economically feasible to make improvements to low-volume rural roads.

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Use of Concrete Median (Jersey) Barriers as Ford Walls in Low Water Crossings

RODNEY F. MENDENHALL AND JOHN R. BARKSDALE

The use of precast concrete median (Jersey) barriers as ford walls on low-volume roads is described. Ford walls are used on U.S. Forest Service roads to stabilize low water stream crossings. This is an acceptable practice on roads that have been temporarily closed for 1 or 2 hours as a result of flooding from sudden and intense storm runoff. The barriers are readily available, precast units that can be transported to the site and installed with conventional equipment that is used to maintain low-volume roads. Modified barriers with steel caps have also been used successfully to prevent erosion of the top of the concrete wall as a result of abrasive bedload movement during high water flows. Ford walls that were constructed with concrete median barriers have been used on hundreds of low water crossings in the desert and mountainous regions of the southwestern United States. These barriers have proved to be an efficient, low-cost alternative to conventional, cast-in-place concrete walls.

The U.S. Department of Agriculture, Forest Service, manages a network of approximately 300,000 mi of road on almost 200,000 million acres of land. These roads are needed to manage a variety of resources and activities, such as timber harvest, recreation, mining, forage, fire protection, and other forest-related activities.

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The Southwestern Region of the Forest Service manages approximately 46,500 mi of road in the states of Texas, Oklahoma, New Mexico, and Arizona. About 70 percent of these roads are of a low standard; most are single-lane roads with dirt or pit-run surfacing and an average daily traffic count of less than 50.

The terrain in the Southwest varies from low Sonoran desert to mountains that are more than 10,000 ft high. Annual rainfall varies from 5 in in the desert to 50 in in the upper pine and alpine forests. Sudden, intense rainstorms are common during the rainy season in July and August. Stream beds that are normally dry become raging torrents within a matter of minutes. These storms often cause injuries and occasionally cause deaths.

A wide range of soil types exists in the Southwest. Much of the soil is composed of highly erodable sandy clays, decomposed granite, and plastic clays. Many roads are impassable for short times during storm runoff because the crossings are flooded and road surfaces are soft and slippery. Good drainage is the key to reducing repair and maintenance expenditures.

THE PROBLEM

The Southwestern Region of the Forest Service has experienced seven major storms since 1972 that have caused over $25 million in damage to roads. Much of the damage occurred at low water crossings. Because of budget restrictions and low traffic counts, drainage structures such as large culverts and bridges are the exception instead of the rule.

The ford structures at low water crossings typically consist of cast-in-place ford walls that are placed in the downstream shoulder of the road to stabilize the road grade at the crossing. These walls are approximately 3 ft high and 6 in thick and may have a footing. Most of the structures were constructed by miners, ranchers, loggers, and road maintenance crews, often with substandard materials. Most of the structures were not designed in accordance with good engineering practices. The walls were subsequently damaged by the movement of rocks in the stream bed. The ends of the walls were also undermined and scoured as a result of undersizing.

Past repair methods involved extending or replacing the wall with a reinforced cast-in-place concrete wall. A 3/8-in-thick steel plate was attached to the tops of some walls to prevent bed load movement from eroding the concrete during floods. Quality concrete is available in this region, but haul distances of 50 mi or more are common. The excavation, forming, mixing, and curing of concrete and dewatering in live streams posed difficult construction problems.

AN ACCEPTABLE ALTERNATIVE

Forest Service engineers faced with tight time schedules for storm damage repairs studied alternate ways to reduce the costly repair and maintenance of ford crossings. The use of a system of precast median (Jersey) barriers as ford walls was a solution to this problem. These precast sections are widely used as temporary traffic separators during construction on state highways and are readily available in new and used condition (see Figure 1).

Jersey barriers are available in lengths of up to 20 ft. A standard section 12 ft, 6 in in length and about 5,000 lbs in weight was selected. This length can be easily transported and handled with equipment that is typically used in the construction and maintenance of low-volume roads.

Design

The structure of the ford wall consists of a series of precast concrete barriers that are embedded at a right angle to the stream and pinned and tied together with a length of 9/16-in cable and clamps (see Figure 2). The end sections are typically sloped upwardly to form a weir shape that forces the water to flow over the center of the structure. The center section is designed to be level.

A 1-ft-wide strip of 70 to 100 equivalent opening size (EOS) geotextile fabric is placed over each joint. The fabric will therefore retain fine-grained material on the upstream side of the wall and prevent holes from developing in the road grade.

The drainage structure should be sized to pass the anticipated storm runoff. Forest Service engineers perform a hydrological analysis for 50- and 100-yr stream flows for major structures. Structures of this type are sized for a 25-yr flow by using an appropriate weir formula or Manning's equation.

The structure is protected with riprap or gabions downstream, as required (see Figure 1). This process is described in many other publications. A 3/8-in-thick steel cap is placed on the structure to prevent bed load movement (boulders) from eroding the top of the concrete. This is easily done at the casting yard because the structures are cast upside down and a C6 × 10.5 steel channel is laid in the bottom of the form and anchored to the concrete.

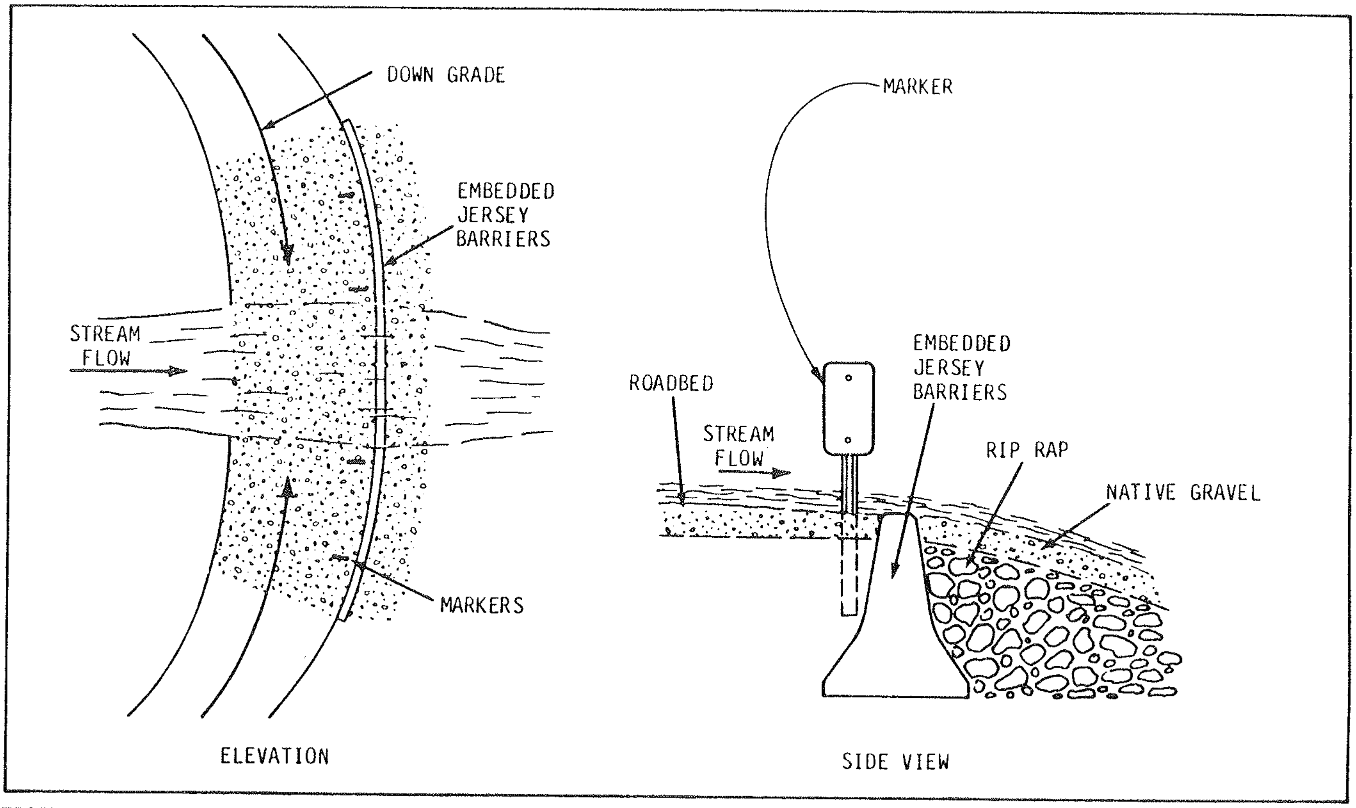

FIGURE 1 Typical installation of barriers.

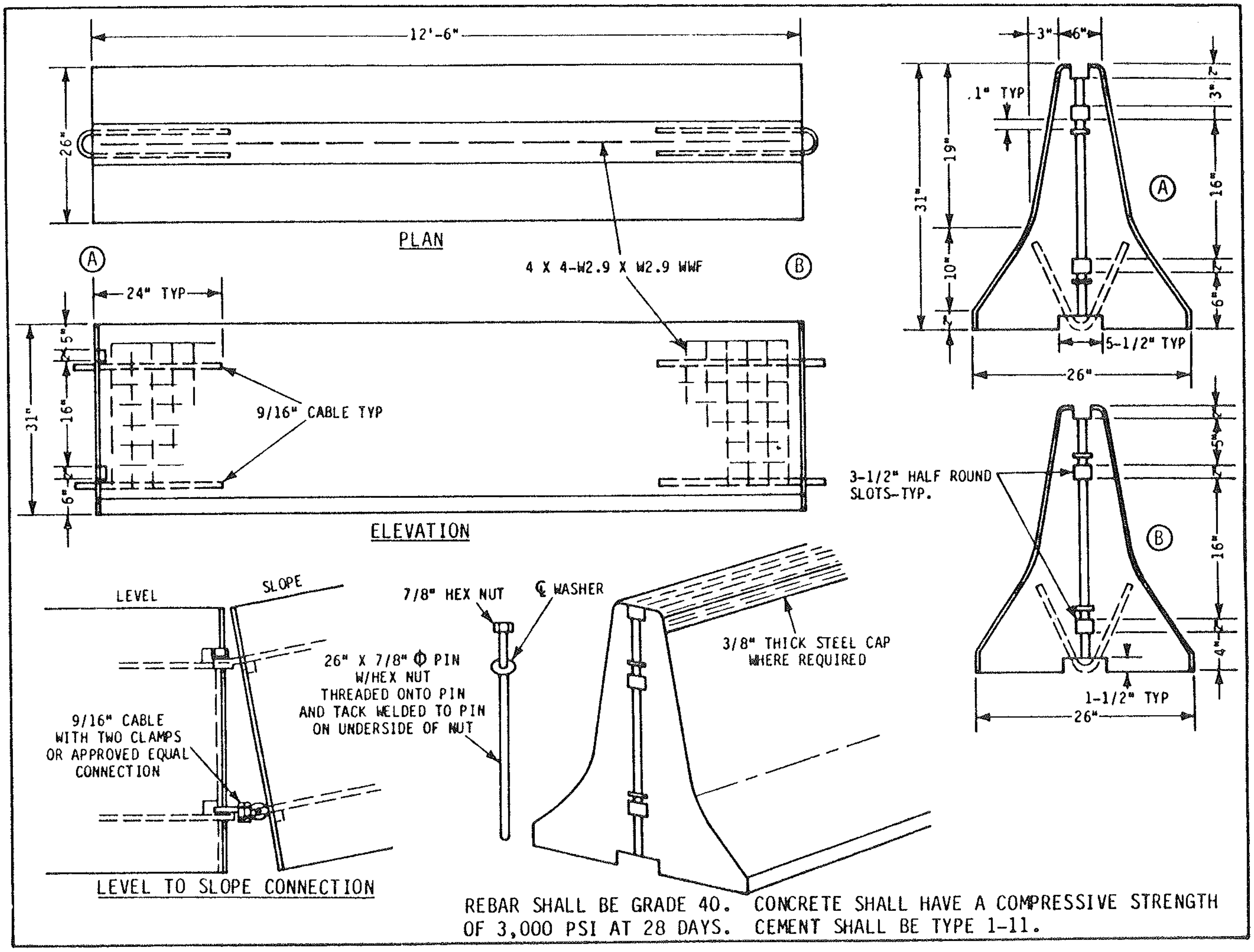

FIGURE 2 Typical fabrication details.

Installation

The equipment needed to install the structures consists of a rubber-tired backhoe and a boom truck or front-end loader large enough to handle the 5,000-lb load.

A trench is dug to the designed shape of the weir. The 12-ft, 6-in sections are then set directly into the trench and pinned or tied together with cable (see Figure 2).

CONCLUSIONS

The following benefits were realized by using precast concrete barriers as ford walls:

- They can be installed in intermittent or live streams without the complicated dewatering or diversion of water required for cast-in-place walls;
- The problem of transporting, forming, mixing, placing, and curing concrete at remote sites is eliminated;
- The foundation requirement is reduced because the base of the barrier is wide;
- Precast walls can usually be salvaged and reinstalled if they are washed out in extreme floods; and
- The purchase and installation costs are reduced from a range of $100 to $200 to $40 to $50/linear ft.

The use of precast concrete median barriers is recognized by the Forest Service as an economical, timely, and effective method of stabilizing ford crossings on low-volume roads.

Culverts Versus Dips in the Appalachian Region: A Performance-Based Decision-Making Guide

RONALD W. ECK AND PERRY J. MORGAN

Based on a literature review and field survey, specific factors that need to be considered in the decision to use culverts or broad-based dips for cross-drainage on low-volume roads were identified. Detailed roadway and environmental information was collected at 19 field sites in the Appalachian region to assess the performance of dips and 18-inch aluminum pipe culverts under a variety of conditions. Performance was rated as either acceptable or unacceptable by a survey team that made a field examination of the drainage structure. Overall, 227 culverts and 255 broad-based dips were assessed. Failure rates for culverts and dips were 7.5 percent and 27.5 percent, respectively. Distress types noted for the culverts rated as unacceptable were sloughing of the cut slope, clogging of pipes and inlets, and erosion of the fill slope. The most common distress types for the dips rated as unacceptable were erosion of the fill slope, rutting, siltation, and ponding. A number of specific conclusions regarding the design and location of dips and culverts were presented to document cases in which one device was more appropriate and cost-effective than another. A decision-making framework, in the form of a flowchart, was developed to assist engineers and foresters in selecting the appropriate drainage device for a particular application.

Drainage is one of the primary concerns in locating and designing low-standard roads that may serve only 0 to 50 vehicles per day (vpd). Drainage must always be adequate if a road is to remain usable. Several types of drainage devices are used to control water flow. Probably the most common type of device is the culvert, which is a closed conduit that carries surface water across or from the road right-of-way. Another device is the broad-based dip, which is a depressed out-sloped section of roadway that acts as a water catchment and drainage channel. Dips can be used instead of culverts for cross-drainage in locations in which no intermittent or permanent streams are present. The plan and profile of a typical broad-based dip are shown in Figure 1. The dip under discussion here should not be confused with low water stream crossings that are frequently found on paved low-volume roads.

Some controversy currently exists regarding the relative benefits and costs of dips and culverts. Some suggest that metal culverts are superior for most drainage needs (*1*, *2*). The initial cost of culverts is high compared with simple drainage devices, but culverts have long lifetimes, require relatively little maintenance, and are essentially unnoticed by road users.

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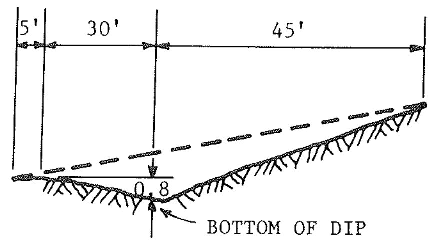
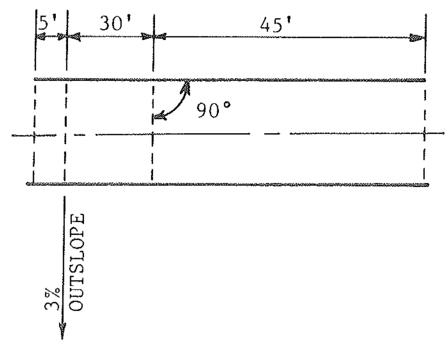


(a) Profile



(b) Plan

FIGURE 1 Plan and profile of a broad-based dip currently used by USFS national forests in North Carolina.

Others promote broad-based dips because they have several advantages (*3*, *4*). Dips have a relatively low initial cost, and unlike culverts, dips can be used without the expense of a ditch line. It has been reported that properly constructed dips have low maintenance costs and, like culverts, do not increase wear on vehicles or reduce hauling speeds (*3*, *5*). However, one disadvantage of broad-based dips is that equipment operators need special training to be able to construct dips properly. Therefore, dips are often not built according to intended specifications.

Design criteria have been established for both broad-based dips and culverts, although actual device dimensions and other details may vary from one geographic region to another. Most drainage devices, if constructed according to specifications and if placed at an appropriate location, will perform satisfactorily for many years. However, if the device is not built according to specifications or it is not properly located, serious problems can result.

An improperly placed or poorly constructed culvert could result in clogging of the pipe or erosion of the roadway or fill slope. Both of these situations can generate siltation and sedimentation, which would consequently degrade the forest vegetation and water quality. A clogged culvert not only increases the likelihood of the roadway washing out in the vicinity of the structure, but it may also introduce the possibility of damage to down-grade drainage structures.

A poorly constructed dip can result in a number of problems, including erosion, siltation, rutting, or ponding of the dip or roadway. Erosion necessitates actions to protect the fill slope and immediate down-grade streams, and replace lost material. Siltation calls for the removal of debris (mostly soil and rock particles) from the dip. Rutting and ponding often require that the dip be reconstructed, because these two types of distress create a build-up of mud or water that eventually creates an impassable roadway. If allowed to continue unabated, the economic costs associated with all of the problems just mentioned can be quite high.

Based on a limited field survey and discussions with forest road designers, builders, and users in the Appalachian region, it became apparent that whereas dips and culverts each have their place as a drainage device on low-standard roads, there are certain conditions in which one is more appropriate than the other. However, no formal engineering study had ever apparently been made of this issue. A need exists to objectively determine through the use of actual field data and an engineering economic analysis, under what conditions conventional metal culverts are more appropriate than broad-based dips on logging roads in the Appalachian region.

STUDY OBJECTIVES

A research project was undertaken to answer this question. Several specific objectives were established to address the overall goal:

- Based on a literature review and field survey, specific factors were identified that had to be considered in the decision to use culverts or dips.
- An experimental design was to be developed to collect detailed data at a number of field sites in the Appalachian region to assess the performance of dips and culverts under a variety of conditions. It should be noted that only aluminum culverts were considered in this study, because they were the only type of drainage device used in the Monongahela National Forest in West Virginia, where the field investigation was conducted.
- An economic analysis was to be conducted of 18-in aluminum culverts and broad-based dips, in which construction, maintenance, and road user costs were considered. The results of the economic analysis are presented elsewhere (*5*).
- Based on the field study and the economic analysis, specific conditions were to be recommended under which culverts or broad-based dips should be installed.

DATA COLLECTION

The factors involved in the performance of a drainage device must be known in order to develop guidelines to assist in the selection of the most appropriate type of cross-drainage. Based on a literature review, field survey, and discussions with practitioners, those factors that affected performance and that needed to be considered in the selection of a dip versus a culvert were identified. In order to establish a convenient framework for decision-making and to formulate the experimental design for the field study, the factors just mentioned were grouped into several major categories, as shown in Figure 2.

In order to determine specific situations in which one drainage structure was more appropriate than another, a field study was designed to evaluate the performance of a large number of broad-based dips and 18-in aluminum pipe culverts in the Monongahela National Forest. The experimental plan was formulated in such a manner that the effects of the variables that influenced dip and culvert construction, performance, and maintenance could be assessed.

Sites were sought that would provide variety in some of the factors, but be similar in others so that the effects of individual variables on drainage device performance could be isolated.

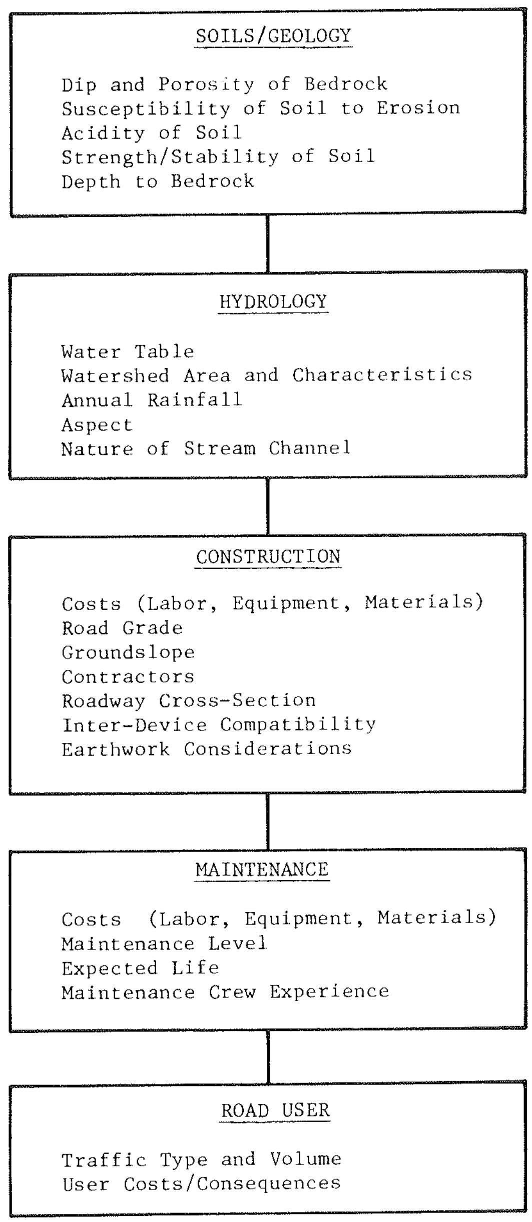


FIGURE 2 Preliminary framework of factors to consider in the selection of broad-based dips or culverts.

Another criterion was that plans and drawings had to be available for the roads selected for study. It was also thought to be desirable that the study roads be associated with active timber sales, although this was not mandatory.

Given these criteria, the investigators met with engineers, soil scientists, and timber sales personnel from the Monongahela National Forest Headquarters in Elkins, West Virginia, to discuss specific study sites. Although a large number of sites were proposed by U.S. Forest Service (USFS) personnel, time and resource constraints limited the actual number to 19. A summary listing of the characteristics of each site is provided in Table 1. Plans, drawings, and appropriate U.S. Geological Survey (USGS) quadrangle sheets were acquired for each of the study sites. Average annual precipitation was estimated from published information (*6*). These items were used to assist in planning the field work and to supplement the data acquired in the field.

Once the field sites were identified, efforts centered on the selection of specific data items to be collected in the field. Information was sought in regard to each study site as a whole and to the individual drainage structures that comprised a site. Data pertinent to the overall site included average annual rainfall, general soil conditions, length, number of broad-based dips, number of 18-in aluminum pipe culverts, traffic volume data, timber sales data, and actual project construction and maintenance costs. Both average daily traffic (ADT) and total number of truck trips necessary for timber removal were sought in the traffic category. The number of truck trips could be computed given the volume of timber sales and the typical number of board feet of timber per truck load.

Information pertinent to individual drainage structures included spacing from a previous structure, type of upgrade structure, road gradient, roadway cross-section, ground slopes, aspect, watershed location, fill height, presence of cut-fill transitions, cross-drain purpose, road surface type and depth, horizontal curvature, and structure performance. Actual device construction and maintenance costs were sought whenever possible.

A study procedure was developed to acquire the desired data efficiently. This procedure basically consisted of a two-person field party that walked over the roads of the study site, recorded the previously mentioned qualitative roadway and environmental information, and made simple measurements to quantify certain roadway and drainage parameters. About 3 weeks were required to complete the structural performance field study.

Two types of data collection forms were prepared and tested to document the information collected in the field. The first form consisted of a summary of information that related to the

TABLE 1 CHARACTERISTICS OF ROADS IN MONONGAHELA NATIONAL FOREST THAT WERE SELECTED FOR FIELD STUDY

Project Sites (USFS Road Name and Number)	Length (miles)	Number of Broad-Based Dips	Number of 18-inch Aluminum Culverts	Avg. Annual Rainfall (Inches)
1. Stuart Camp, 91-B	0.09	2	1	49.0
2. Stuart Camp, 91-C	0.43	9	0	49.0
3. Suart Camp, 91-D	0.32	6	1	49.0
4. Stuart Camp, 319	0.10	2	0	49.0
5. Peach Orchard, 297	0.77	6	1	38.7
6. Four Mile, 969	1.31	14	11	45.7
7. Sue, 796	0.63	10	0	45.7
8. Sue, 796-C	0.95	11	7	45.7
9. Music Run, 907	1.22	0	20	59.7
10. Hacking Run, 914	0.52	6	7	56.5
11. Galford Run, 90	3.16	0	41(15)[a]	47.1
12. Galford Run, 90-A	3.30	39(15)	16(5)	47.1
13. Stony Run, 757	1.54	17	13	47.1
14. Divide, 790	1.77	0	8	47.1
15. Leatherwood, 368	6.52	34(7)	54(13)	54.6
16. Jobs Run, 117	2.35	46	13	53.8
17. Lick Drain, 929	4.40	47	29	53.8
18. Red Run, 244	3.60	0	67	53.8
19. Warner Run, 916	3.64	63	16	50.1

[a]Numbers in parenthesis represent number of drainage structures studied for those instances where it was not possible to survey all structures.

overall project. One form was completed for each project. The form was designed to be completed by Forest Service personnel and the researchers (using the maps, plans, and drawings furnished by the Forest Service) before the field surveys were conducted.

The second data form was completed in the field by the two-person study team. Information about the location and environment of each individual drainage structure was recorded. One form was used for each structure. It should be noted that because of time, resource, and weather constraints, a complete set of data could not be acquired for all sites.

Performance was rated as either acceptable or unacceptable by the survey team after completing an examination of the drainage structure. Unacceptable structures were those that required, in the opinions of the field party, immediate maintenance attention. When a structure was determined to be unacceptable, a note was made of its distress type, which could have been one of the following:

- Rutting of dip or roadway,
- Siltation of dip,
- Erosion of fill slope,
- Corrosion of pipe,
- Ponding of dip,
- Clogging of inlet or pipe,
- Sloughing of cut-slope, or
- Construction problems.

Before the field work was begun, special efforts were made to ensure that the survey team understood the types of distress. Distress types were defined both in words and by photographs (taken during the field trips) that depicted an example of a particular condition.

The format of the data on soils, which consisted of such items as soil name, geologic formation, soil erosion potential, and other information, was developed based on unpublished USFS guidelines of soil characteristics for drainage and road building in the Monongahela National Forest. The soil found at each cross-drain was identified by use of a combination of descriptions of soil color and texture made in the field, and information from USDA county soil surveys and USFS documents. Once the soil types and geologic formations were identified, the USFS rating guide for soil sensitivity groups in Monongahela National Forest was used to determine relevant soil characteristics. However, the coarse fragment content of the soil was determined visually. Estimates of depth to bedrock and depth to seasonal high water table had to be obtained from USDA county soil surveys.

DATA ANALYSIS

The data were initially categorized in such a manner that the performance of 18-in metal culverts could be examined separately from the performance of broad-based dips. Of the 482 structures studied, 227 were culverts and 255 were broad-based dips. When study sites were selected, an attempt was made to examine an approximately equal number of dips and culverts. Seventeen of the 227 culverts were rated as unacceptable, which represents a failure rate of 7.5 percent. The overall failure rate for dips was 27.5 percent. The performance of the culverts that were studied was therefore substantially better than that of the dips.

The distress types noted for the culverts that were rated as unacceptable were sloughing of the cut slope, clogging of pipe or inlet, and erosion of the fill slope. The most serious problems were sloughing of the cut slope and clogging of the pipe or inlet. Sixteen of the 17 failures involved sloughing problems. Fifteen of the failed culverts were clogged; only three culverts were noted to have erosion of the fill slope. Sloughing of the cut slope and clogging are related distresses; the material that sloughs off the slope gets deposited in the inlet or pipe and clogs the structure.

The most common distress types for the broad-based dips that were rated as unacceptable were erosion of the fill slope, rutting, siltation, and ponding. Of the 70 failed dips, 48 were noted for erosion of the fill slope, 35 for rutting problems, 34 for siltation, and 27 for ponding. Four dips were noted for construction-related distress. Only one dip that failed was noted for sloughing of the cut-slope.

In order to assist in specifically determining which factors affected drainage structure performance, the data were organized into the following groups: design factors, soil and geologic factors, hydrologic factors, and traffic factors. The various factors that comprised these categories were analyzed individually to determine if each had a substantial effect on structure performance.

Substantial factors were those that were judged by the researchers to yield a relatively high failure rate; all other factors were considered secondary. This terminology was arbitrarily selected by the investigators for their convenience in describing the data tabulations; no statistical significance should be attached to the results.

The researchers used a statistical method known as the normal approximation to the binomial to compare the overall performance of culverts and dips with the performance of these devices when categorized by the various factors (*7*). Plots were prepared by use of the appropriate statistical equation and the aforementioned overall structure performance rates for 18-in culverts and broad-based dips. The plots depicted statistical significance as a function of the number of failed and number of total structures within a given sample set. The following three statistical regions were identified:

- Values that indicated a significantly better structural performance when compared to the overall situation,
- Values that indicated a significantly worse structural performance, and
- Values that indicated no significant change in structural performance.

Design Factors

The following items were considered as design factors:

- Road grade,
- Roadway cross-section,
- Structure spacing,
- Road surface type,
- Immediate up-grade (on-roadway) structure,
- Fill height,
- Horizontal curvature,
- Cross-drain skew, and
- Cut-fill transitions.

Appropriate groups of data were developed for each factor and the frequency of device failure was determined for the groups. Roadway cross-section and cross-drain skew were determined not to have a substantial effect on drainage device performance.

An examination of structure performance versus road grade indicated that culverts performed best when the road grade was 7 percent or less. The culvert failure rate was only 2.5 percent for road grades less than or equal to 7 percent. The failure rate was 13.1 percent for road grades greater than 7 percent. Broad-based dips demonstrated a somewhat comparable trend in that they also performed better as the road grade decreased. However, dips tended to perform best when the road grade was less than or equal to 3 percent. The dip failure rate was 32.3 percent for grades between 3 and 9 percent.

Drainage structure spacing is a design factor that depends on the road grade. Mean spacings for acceptable and unacceptable culverts and dips, respectively, are presented in Figures 3 and 4 for different road grades. Those devices that did not have an adjacent drainage structure were deleted from this particular analysis. The spacing of failed structures was generally greater than the spacing of acceptable structures. The data also indicated that the mean spacing for acceptable culverts was relatively close to the design spacing value. The mean spacings for unacceptable dips deviated more widely from the design value.

Structure performance versus road surface type was also examined. As was expected, broad-based dips generally performed better when armored with gravel than they did in an unsurfaced condition. The failure rate for unsurfaced dips was 37.3 percent compared to 19 percent for dips armored with stone. The data were insufficient to evaluate which type of gravel surfacing performed better because only seven dips were armored with 3-in quarry stone. Based on information acquired during practitioner input, there appeared to be some disagreement between engineers as to whether 3/4-in crusher run or 3-in quarry stone was a more appropriate surfacing for logging roads in the Appalachian region. Additional research into this issue could prove to be fruitful.

Culverts or dips located on horizontal curves had a higher frequency of failure than structures located on tangent sections of roadway. However, dip performance appeared to be more adversely affected by curvature than that of culverts. The failure rate for dips located on curves was 40 percent, compared to about an 8 percent failure rate for culverts located on curves.

Fill height is another design factor that affects structure performance. The failure rate generally increased as the height of the fill increased for both dips and culverts. Closer examination indicated that the failure rate increased dramatically for dips when the fill height was greater than 3 ft, as shown in Figure 5. The failure rate for fill heights less than or equal to 3 ft was 18.9 percent, compared to a rate of 67.2 percent for fill heights greater than 3 ft.

Culvert and dip performance versus type of drainage structure immediately up-grade from the one in question was investigated. Drainage structures that were located in a sag were separated from the other structures in this analysis. This was done in order to specifically examine structure performance at sag locations, which practitioners had indicated were locations that were critical to dip performance. The analysis indicated that sag locations had a 45.5 percent failure rate for broad-based dips and a 9.7 percent rate for culverts. Based on these results, it is recommended that if it is necessary to locate a drainage structure in a sag location, a culvert should be used because it will probably perform better than a broad-based dip.

Although it was difficult to determine from the data which type of structure was better in terms of drainage, it was noted that dips were more sensitive than culverts to the existence of an up-grade drainage structure. When no structure was located up-grade from a dip, the failure rate was 41.1 percent. By contrast, when no structure was located up-grade from a culvert, the failure rate was only 8.7 percent.

Notably few study site drainage devices were located at cut-

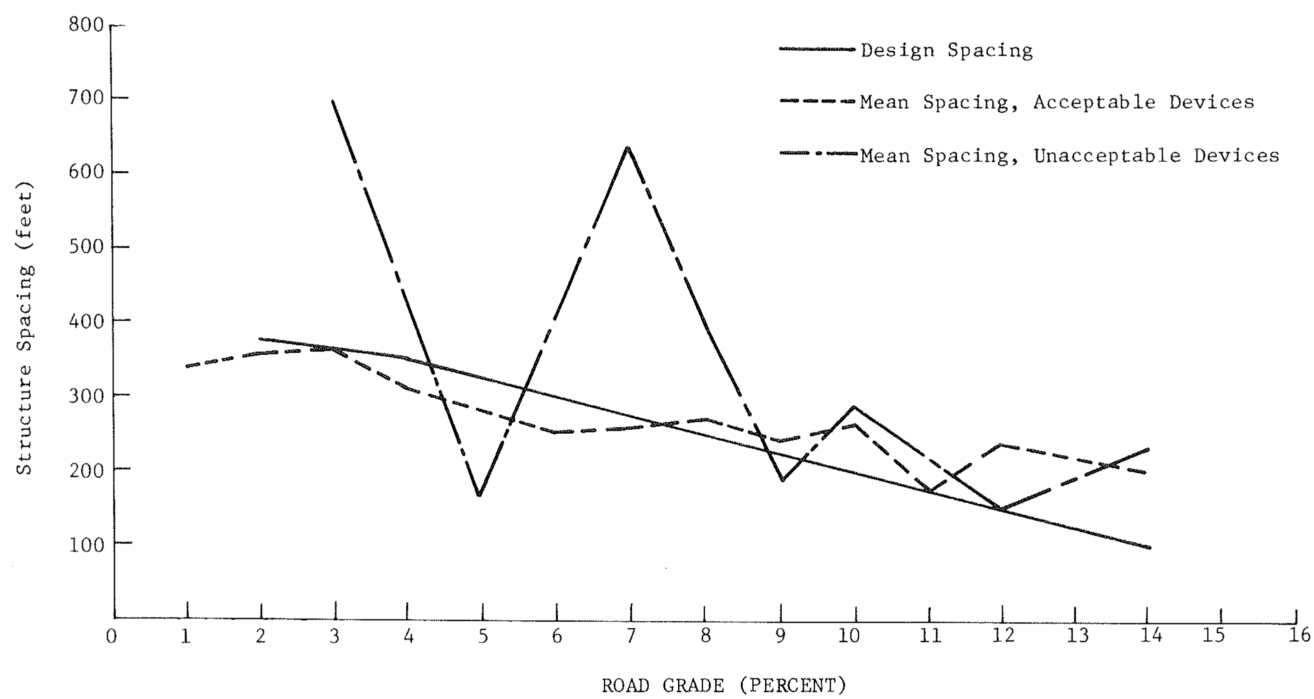


FIGURE 3 Mean culvert spacing versus road grade.

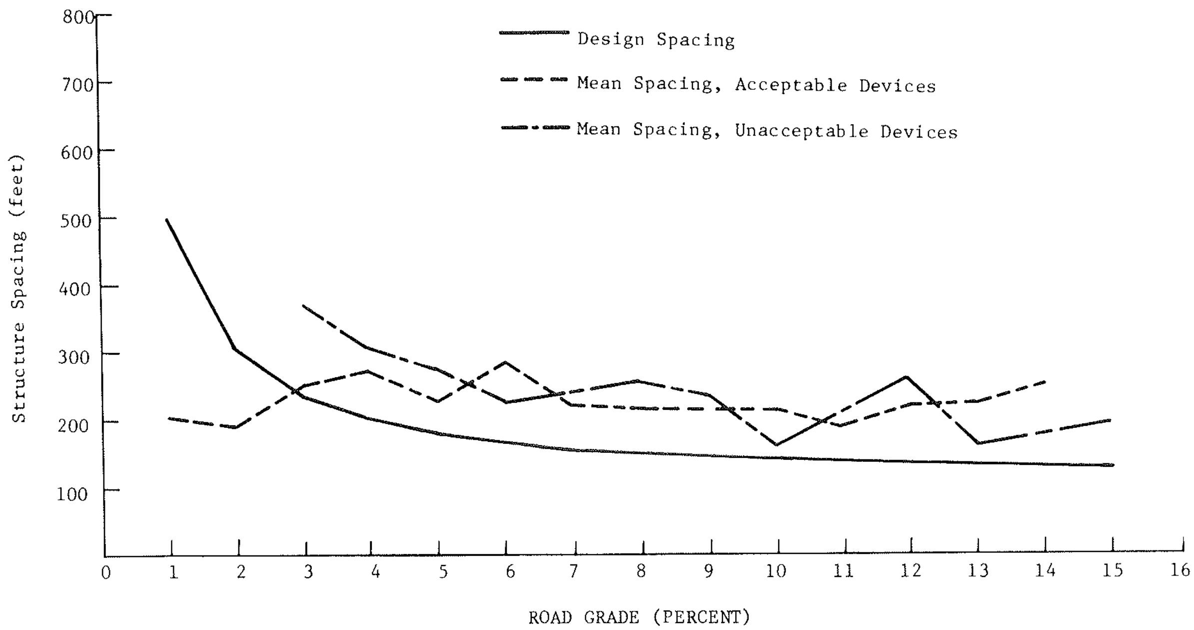


FIGURE 4 Mean spacing of broad-based dips versus road grade.

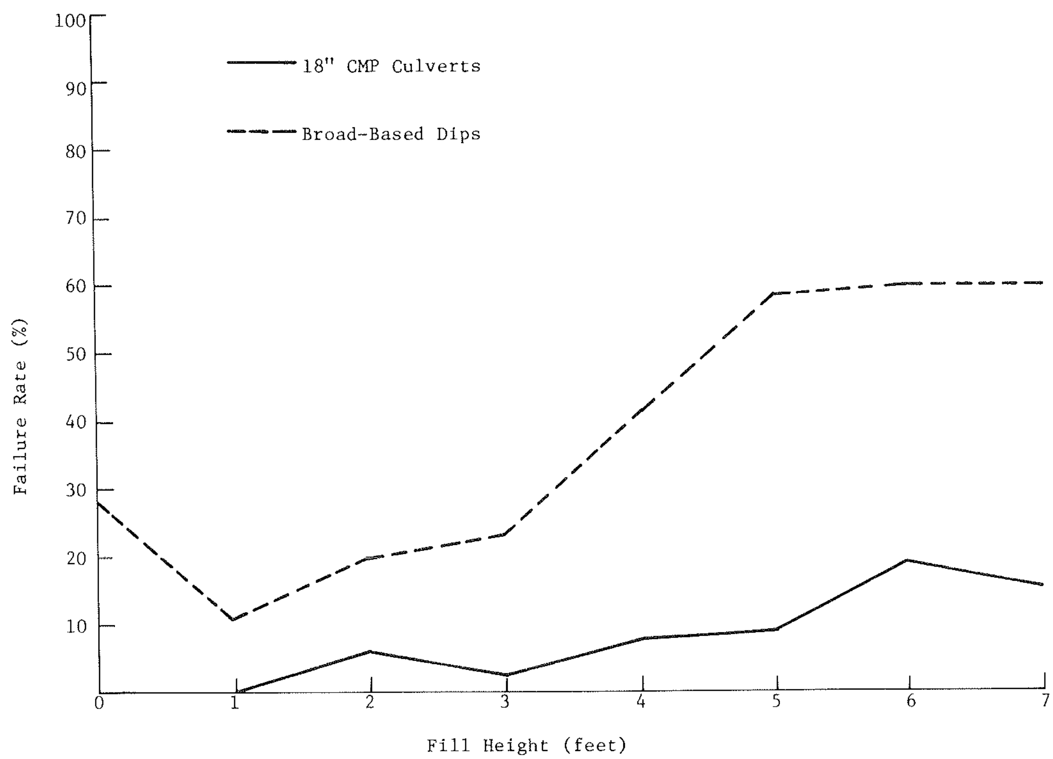


FIGURE 5 Drainage structure performance versus fill height.

fill transitions. This may be an indication that designers have learned from experience to avoid placing drainage structures at cut-fill transitions. Based on a very small sample size, it appears that culverts should be used when drainage structures must be located at cut-fill transitions. Although only four dips were located at cut-fill transitions, two of them failed. However, none of the five culverts at cut-fill transitions failed.

It should be noted that a limitation of the analysis just described was the assumption that all drainage grades, frequency and amount of flow, and other parameters of this nature were the same for the sites under consideration. A comparison of failure rates based on a single variable, such as whether or not an up-grade drainage structure exists, could be misleading unless other flow characteristics are considered.

Soil and Geologic Factors

The following items were considered soil or geologic factors:

- Soil type,
- Geologic formation,
- Soil erosion potential,
- Ground slope,
- Suitability of soil as road material,
- Coarse fragment content, and
- Depth to bedrock.

Soil erosion potential, soil suitability for road building, coarse fragment content, and depth to bedrock were analyzed but were determined not to be significant factors in drainage structure performance.

Structure performance versus specific type of soil was examined initially. However, the number of different soil types studied was so large that it was difficult to determine which soils contributed to acceptable or unacceptable structure performance. Therefore, soils were reclassified according to soil series. For example, any soil type that had Berks in its title was classified as a Berks soil. The performance of 18-in metal culverts was essentially independent of soil series. However, certain soils tended to contribute to poor dip performance. Soils associated with high failure rates for broad-based dips were generally gravel-sand-silt mixtures, gravel-sand-clay mixtures, sand-silt mixtures, and silty or clayey fine sands that were typically derived from interbedded sandstone, siltstone, and shale.

It should be noted that the soil data collected in this study were admittedly general in nature. Laboratory testing would have been desirable to determine specific reasons why the specific soils series had such high failure rates. It is believed that the high failure rates may be related to the fineness of these soils, the poor bearing strength of shale fragments present in several of these soils, and other characteristics of the underlying geologic formation.

Because soil characteristics are closely related to and vary with the underlying geologic formation, an analysis was made of the influence of geologic formation on drainage structure performance. This analysis revealed that culvert performance is independent of geologic formation. Dips, however, demonstrated high failure rates in the interbedded sandstone, siltstone, and shale formations.

The ground slope was also examined because of its suspected relevance to structure performance. Because water is transmitted to drainage structures from higher elevations, it was believed that the ground slope above the drainage structure was critical. Culvert performance worsened as the ground slope increased. However, the same trend was not true for dips.

The poor performance of culverts on steeper ground slopes may result from material such as rocks, limbs, and other unstable materials rolling down these steep slopes and becoming lodged in the culverts. High cut slopes, which are undesirable for culverts, may be needed when the ground slopes are steep. Because dips are not enclosed like culverts, they do not exhibit these characteristics, and are not as dependent on changes in the ground slope as culverts.

Hydrologic Factors

Hydrologic factors affect the quantity of water that passes through a drainage structure. These factors are obviously an important component of dip and culvert performance. The following items were considered hydrologic factors:

- Aspect,
- Watershed location,
- Average annual precipitation,
- Presence of seeps or springs,
- Cove location,
- Soil wetness,
- Surface water yield,
- Ground water yield, and
- Depth to seasonal high water table.

Soil wetness, surface water yield, ground water yield, and depth to seasonal high water table were examined, but no substantial differences between devices were indicated.

A limitation of the data was that the aspect was classified as either north or south to ease the tabulation of data, and because these exposures had been identified by practitioners to be an important determinant of dip performance. However, the classification of aspects into such broad categories could have influenced the results of the study. For example, a drainage structure located on an east-northeast exposure could have received the benefit of the drying effect of solar radiation because its exposure was closer to the east than to the north. However, in this study the exposure would have been classified as a north aspect.

Results indicate that culverts and dips located on a north aspect both had higher failure rates than those with a southern exposure. This can be attributed to moisture- and water-related problems that are exacerbated by the lack of solar radiation and its drying effect. The failure rate for dips with a north aspect was not as great as had been anticipated, which could be a result of the data limitation just described.

Watershed location was another factor that was studied. The location of the drainage device in the watershed was classified as belonging to either the upper, middle, or lower third of the watershed. The purpose of this breakdown was to provide a rough estimate of the relative amount of runoff handled by the device. As the watershed location went from high to low, failure rates increased for both culverts and dips. This suggests that the greater the volume of water handled by the drainage structure, the greater the likelihood of failure.

The average annual precipitation indicates the variability in the quantity of water handled by drainage devices from project to project. The analysis of this factor was made difficult by the fact that rainfall data were estimated using available meteorological information instead of collecting specific data from each site. Two study roads would therefore be assigned the same quantity of precipitation that was determined from the nearby weather station although they were several miles apart and probably received slightly different amounts of rainfall. However, it was believed that the relative, rather than the absolute, amount of annual precipitation would be of greater value in this study.

Culverts and dips both experienced an increase in failure rate as the average annual precipitation increased. This result had been expected, because structures in regions of high precipitation carry large volumes of runoff, and are prey to ground water or moisture-related problems.

Two locations that tend to cause problems for drainage structures were identified from the literature review and practitioner input. Drainage structures located in coves or where springs or seeps are present will be generally exposed to larger quantities of water than they would at other locations. The Forest Service tends to avoid locating dips where springs, seeps, or coves exist. It was the opinion of the researchers that the reason three of the twenty-two 18-in aluminum culverts failed where seeps or springs were present was that these pipes were undersized. A larger pipe would probably have functioned properly in these three locations.

Traffic Factors

One would intuitively expect drainage structure performance to be related to the magnitude of traffic using the roadway. Data were available in this study for average daily traffic and total truck traffic. Values for ADT were determined from estimates furnished by forest rangers for the projects within their particular jurisdiction of the Monongahela National Forest. The data indicated that the volume of traffic did not affect culvert or dip performance. This latter finding was unexpected; however, the available data did not yield a specific explanation. Because broad-based dips are actually part of the road surface, it had been hypothesized that an increase in traffic would correlate with an increase in the failure rate for dips.

Perhaps a better measure of the traffic load applied to a drainage structure is the total number of truck trips made to date on the logging road. The total number of truck trips was derived by dividing the quantity of timber (in board-feet) involved in the timber sale by the average number of board-feet of lumber per truckload. Therefore, in this case traffic volume could be considered a surrogate measure for the weight applied to the roadway. Both the number of vehicles and their weights have an impact on roadway and drainage structure performance.

Culvert and dip performance were not affected by changes in traffic volume. This finding was unexpected. An important factor for which data were not available in this study was the condition of a dip when it received traffic. A dry dip can withstand many more load applications than a wet one. It is hypothesized that the amount of truck traffic could be an important factor in the prediction of dip performance under certain conditions. Additional research is warranted in this area.

THE DIP VERSUS CULVERT DECISION

Information was provided in the preceding section on the performance of metal culverts and broad-based dips in relation to certain design, soil, hydrologic, and traffic factors. An earlier study compared culverts and dips on an economic basis in terms of construction, maintenance, and road user costs (*5*). The findings of each of these aspects in the overall study were combined to develop guidelines that would assist in the identification of those conditions in which aluminum pipe culverts are more appropriate than broad-based dips for intermittent cross-flows on Appalachian logging roads. The results are presented in this section in the form of a guided decision-making scheme. It should be noted that the guidelines (and the research findings from which they were developed) are based on dips and culverts that were constructed to USFS standards. The guidelines are not directly applicable to drainage devices that were not constructed to these standards.

The approach taken was to identify those forest road situations in which broad-based dips were not an appropriate drainage device. By a process of elimination, a drainage location that did not possess any of these characteristics would be a likely candidate for the installation of a broad-based dip. A list was developed of conditions that had been identified in the drainage device performance study as being strongly associated with poor dip performance. The traffic volume criteria from the economic analysis and certain other conditions that had been identified from practitioner input were added to this initial list. This list of conditions formed the decision-making framework for the dip versus culvert decision. A review of the list indicated that a flowchart might be the most appropriate format for presenting the guidelines on dip and culvert usage. The decision-making flowchart is shown in Figure 6.

The user begins the decision-making process by identifying the drainage location of interest on a logging road and noting its characteristics. Characteristics for which information is needed include:

- Road surface type,
- Volume and type of vehicular traffic,
- Hauling schedule,
- Stream characteristics,
- Water table characteristics, and
- Roadway design elements, such as curvature, grade, and cut-fill characteristics.

By considering one drainage location at a time and answering "yes" or "no" to questions about the characteristics just noted, the analyst follows the flowchart until one of two possible outcomes is reached: (a) a culvert is recommended for the particular location or (b) use of a broad-based dip appears feasible. In the latter case, a soil scientist or geotechnical engineer should be consulted for advice on the suitability of the soil for dips. Although it was initially intended to develop specific soil and geologic criteria for dip installation, this proved to be impossible for several reasons. Although a variety of soil and geologic combinations had been examined in the field study, insufficient data existed about any single combination to permit firm conclusions to be drawn. The results were similarly limited because they were based on a relatively cursory field observation of soil characteristics instead of on a more thorough or quantitative analysis of soil properties. Finally, the type of soil data the typical user would have available was not known.

The traffic volume criteria that appear in the guidelines were based on results of the economic analysis (*5*). It is recommended that culverts should strictly be used on any road that carries an excess of 15 vpd no matter if it is subjected to this traffic level for 2 or 20 years, because of the high user cost associated with dips. Dips are appropriate on roads with traffic volumes less than 5 vpd, assuming that their use is not precluded by a design, soil, or hydrologic factor. For traffic volumes between 5 and 15 vpd, the decision to use a dip or culvert should be influenced by how much the road is used by log-hauling vehicles. If the road is used each year for the life of the road (assumed to be 20 yrs), the high road user costs associated with broad-based dips make culverts the preferred drainage device. In cases in which the road is to be used for timber harvesting only during the first few years of its life and then closed for a period of time, broad-based dips are the more economical drainage structure.

Once the decision of whether to use a dip or a culvert has been made, the user should repeat the process by identifying the next

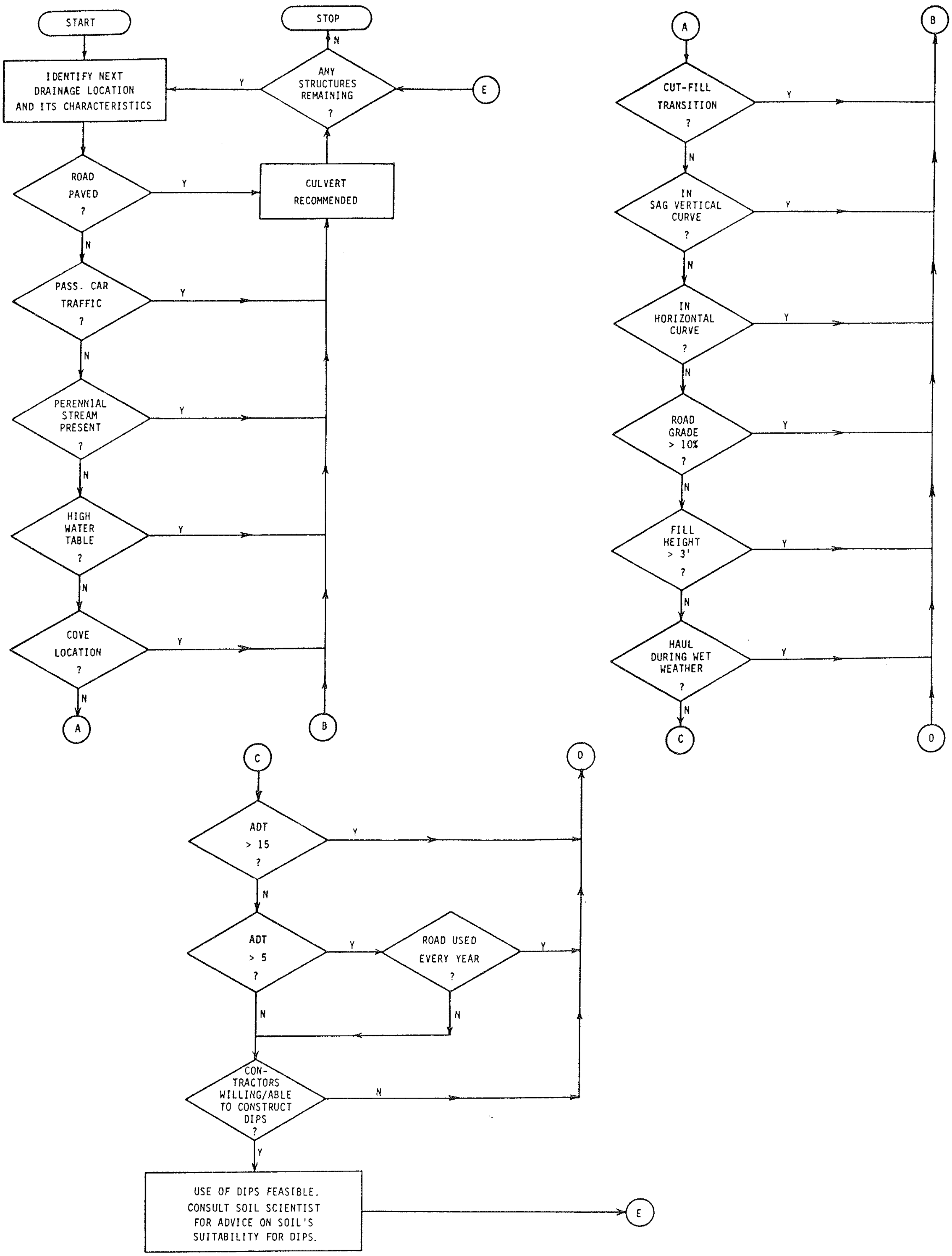


FIGURE 6 Decision-making flowchart for the selection of either broad-based dip or 18-in aluminum culvert for use on forest roads in the central Appalachian region.

drainage location on the road and proceeding through the list of questions again. This procedure should continue until all drainage locations are evaluated. It should be noted that the decision-making framework presented here is based on the assumption that the optimal design of the road will include a mix of dips and culverts. There may be certain situations in which a road may be built in which dips or culverts are used exclusively for legitimate reasons. For example, if local contractors have neither the ability nor the desire to build broad-based dips, all drainage devices on the road would be culverts.

It must be emphasized that the flowchart described earlier is only a guide or aid in the selection of the type of drainage to use on forest roads. The user's experience and familiarity with haul road design and drainage, and with the region in which the road is located, will play a major role in determining how effectively the flowchart meets its intended objectives. It must also be kept in mind that the development of the flowchart was mostly based on data from the central Appalachian region. Users in other regions of the country may find certain items on the flowchart inappropriate or may feel a need to include an additional decision-making capability. Such modifications can be handled relatively easily and would increase the flexibility of the flowchart as a decision-making tool. For example, the flowchart presented here applies to 18-in, corrugated aluminum pipe culverts. If steel culverts are being considered, it might be appropriate to check the pH of the soil. An acidic soil could significantly shorten the life of a steel culvert.

CONCLUSIONS AND RECOMMENDATIONS

Neither dips nor culverts are a panacea for drainage problems on logging roads; each device has its unique strengths and limitations. However, certain situations exist in which one device may be more appropriate and cost-effective than the other. These conditions have been documented, and a decision-making framework was developed to assist engineers or foresters in selecting the appropriate drainage device for a particular application.

The performance of the culverts in this study was substantially better than that of broad-based dips. Many of the dip failures could be traced to one or more underlying factors that, in retrospect, made the installation of a culvert the more appropriate solution for that location. Culverts, however, generally failed as a result of the pipe or inlet clogging. This demonstrates the importance of a regular culvert inspection and maintenance program to identify and correct problems before they reach destructive levels. Other, more specific conclusions that were drawn from the drainage structure field performance study are as follows:

- Drainage device spacing guidelines that were found in the literature are appropriate for the central Appalachian region. Wider spacing of dips and culverts to reduce costs is not recommended because failure rates increase dramatically.
- Dips armored with gravel perform better than unsurfaced dips.
- Dips should not be installed at horizontal curves.
- Dips should not be installed on fills more than 3 ft high.
- Culverts are preferable to broad-based dips in sag locations.
- The performance of 18-in, corrugated metal pipe culverts is relatively independent of soil characteristics. Dip performance, however, is closely related to the soil's erodibility and other characteristics. The advice of a soil scientist should be sought before a broad-based dip is recommended for a particular location.
- Dip performance is independent of ground slope. Culverts, however, are prone to clogging when ground slopes exceed 45 percent.
- Culverts located on road grades of less than or equal to 7 percent should perform effectively if they are constructed and maintained in accordance with recommended USFS guidelines.

The results of this study unexpectedly indicated that culvert and dip performance was not affected by traffic volume. An important factor for which data were unavailable was the condition of a dip when it received traffic. A dry dip can withstand many more load applications than a wet dip. It would be desirable to extend the results of this study by examining the relationship between traffic volume, dip condition (wet or dry), and structural performance.

Although the results of this study were based on very general soil data, they demonstrated the important role that soil and geologic factors play in the prediction of drainage structure performance. An additional, in-depth study by a soil scientist or geotechnical engineer of specific soil and rock types and their relation to drainage structure performance is warranted.

Broad-based dips armored with gravel generally perform better than unsurfaced dips. The data were insufficient to evaluate which type of gravel surfacing performed better because only a few dips were armored with 3-in quarry stone; the rest were surfaced with crusher-run stone. Based on information acquired during practitioner input, there appeared to be some disagreement between road designers as to whether 3/4-in crusher-run stone or 3-in quarry stone was a more appropriate surfacing for logging roads in the Appalachian region. Given the current high cost of road surfacing materials, additional research into this issue could be fruitful.

ACKNOWLEDGMENTS

This paper is based on research supported by funds provided by the U.S. Department of Agriculture, Forest Service, Northeastern Forest Experiment Station. Special thanks are extended to Randall Burks, who played a major role in the field data collection. Appreciation is also expressed to the staff of the Monongahela National Forest for their assistance and cooperation throughout the study.

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Rock Riprap

There are three basic types of riprap: dumped, hand-placed, and grouted. The dumped or hand-placed stones constitute a protective lining that is composed of multiple layers of stones that rest on the foundation soil or a bedding layer. The multiple layers ensure that the underlying soil is not exposed if settlement occurs or if scouring by ice or debris occurs.

In terms of cost, the best alternative is dumped riprap, which requires less labor cost. Grouted riprap is the most rigid material and the most susceptible to failure by undermining. Dumped riprap is the material that is least vulnerable to impact damage.

The size and grading of rocks to be used are important. A well-graded riprap acts as its own filter layer and prevents outwash of the underlying soil. A well-graded riprap can be thinner than a uniformly graded riprap with a special filter layer.

Soil Cement

Soil cement can be used as a substitute for riprap. This is especially useful where suitable stone is not available or is costly. Soil cement blocks can be cast at the site and hand-placed to guard against erosion. Soil cement is relatively inexpensive and portions can be replaced with ease. The labor of casting and hand-placing the blocks can be significant.

Soil, sand, and cement have been used to form an erosion-resistant surface. It must be placed under dry conditions and compacted. Shrinkage cracking and a low flexural strength may create problems.

Gabions

Gabions are wire baskets that are filled with stones. They have been used successfully on low water crossings. Reno mattresses and Fabriform are also examples of commercially available slope protection materials.

Gabions have the advantage of being flexible, which makes them less prone to settlement or undermining. They also fill up with silt and can support vegetative growth. Gabions are also usually cheaper than concrete. Suitable rock filler material must be available.

Reinforced Concrete

Reinforced concrete is the most elaborate and costly form of protection; it is also the most durable and requires the least maintenance costs. Designers must consider the use of suitable reinforcement to guard against undermining and scour.

Adjacent Erosion

When selecting a site, the designer should select a location where the stream is stable. If evidence of aggradation, degradation, or lateral migration is evident, an attempt should be made to relocate the crossing or provide remedial measures.

If the designer determines that erosion adjacent to the crossing may occur, erosion-resistant materials or cut-off walls should be provided. The exit velocity, depth of scour, and length of stilling basin must be estimated.

Seepage Considerations

Two potential problems can arise as a result of subsurface seepage beneath hydraulic structures: excessive uplift pressures and piping. The probability of these problems increases with an increasing head difference between the upstream and downstream sides of the crossing. The difference in head may not be large in vented fords, whereas head differences of more than 2 ft might occur in a case in which a ford is used. A flow net analysis was performed using typical ford geometries and sediment properties for a 2-ft head difference. This analysis indicated that, without any cut-off for seepage control, it is unlikely that problems of excessive uplift pressures and high exit gradients will occur; cut-offs for seepage control would therefore be unnecessary. However, if the designer anticipates unusual conditions, a flow net analysis should be conducted to evaluate both pore pressure distribution and exit gradients for conditions of no cut-off and various cut-off geometries.

Although a cut-off may not be justifiable as a means of seepage control, it may be necessary as a protection against scour. The presence of a cut-off wall on the downstream side of a low water crossing will have the effect of decreasing seepage quantities and decreasing exit gradients relative to a condition of no cut-off. However, the cut-off will have a tendency to increase uplift pressures on the downstream side of the crossing. Therefore, it is recommended that if a cut-off is designed for scour control, the structure should be analyzed with a flow net to ensure that the pore pressures are not excessive.

CONSTRUCTION DETAILS

A detailed construction procedure is not practical because of the wide range in the variables of materials and site characteristics. However, certain elements of construction have been successfully used and are included here as examples.

The various components of an LWSC are shown in Figure 2. The design elements were described earlier. The use of cables to hold pipes in place in case the core material is washed out is shown in Figure 7. Examples of side walls and cut-off walls are depicted in Figure 8. These devices are used to protect the edges of the crossing and to prevent erosion of the core filler material. An example of erosion protection for a high type of crossing is shown in Figure 9. The extent to which crossing material can be provided is depicted in Figure 10. Different types of unvented ford protection are shown in Figure 11.

TRAFFIC CONTROL

A low water stream crossing has two unique characteristics that are not associated with a traditional bridge. The vertical profile at the crossing is usually restricted to low speeds and the pavement surface is subject to periodic flooding. Adequate warning of these conditions should be provided to the user. The following recommendations are based on recent research by Carstens and Woo (*4*).

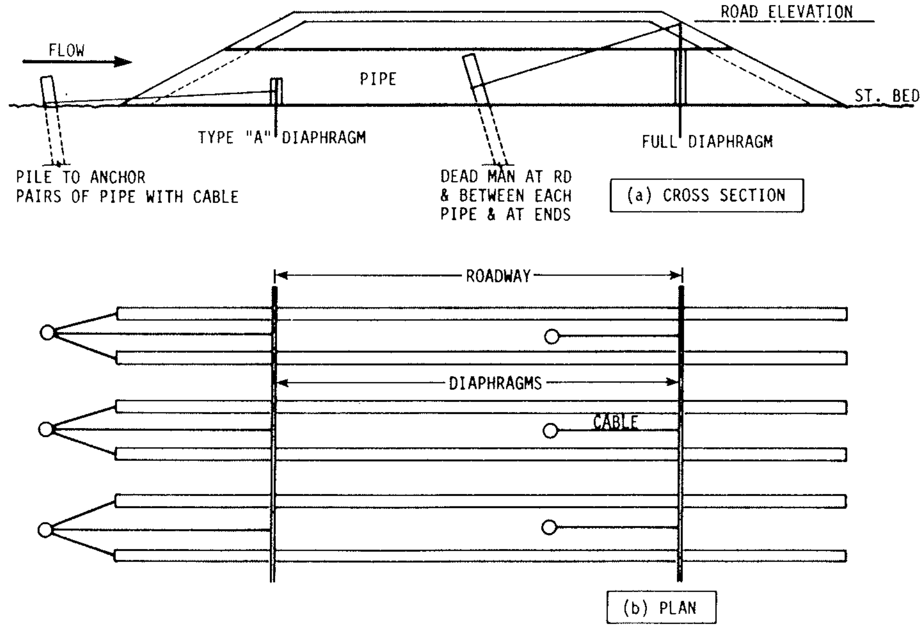

FIGURE 7 Cable anchor details.

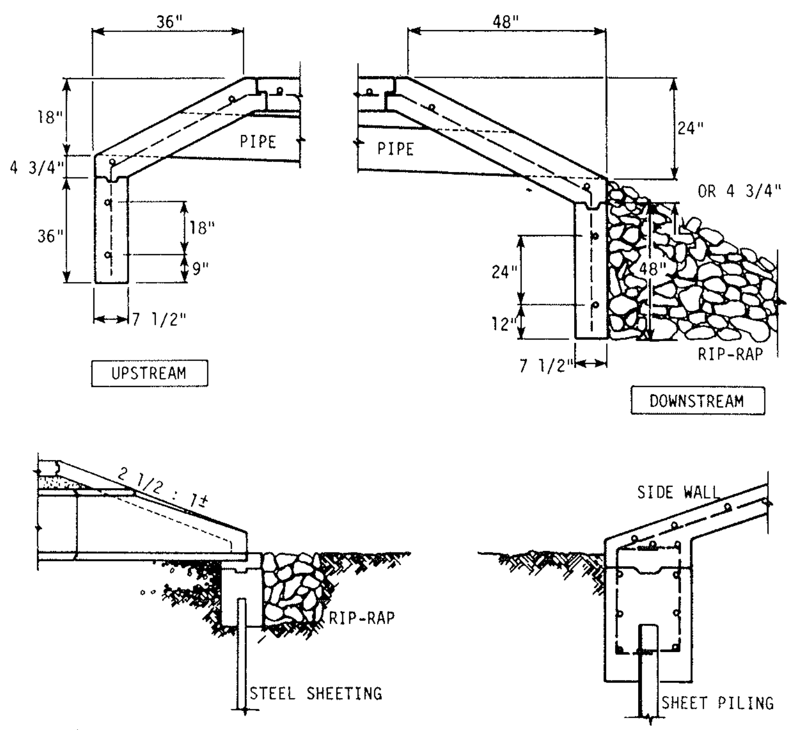

FIGURE 8 Typical sidewall and cut-off wall sections.

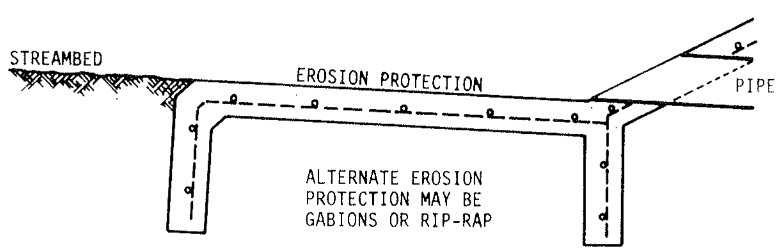

FIGURE 9 Typical erosion protection.

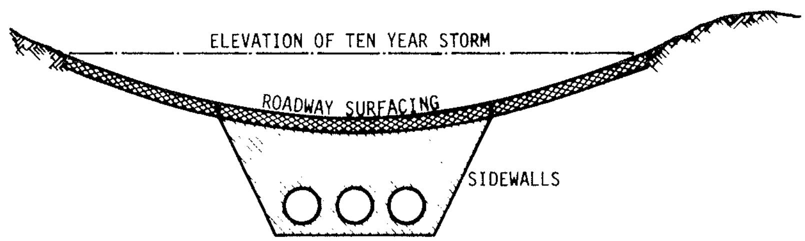

FIGURE 10 Minimum limits of LWSC roadway surfacing.

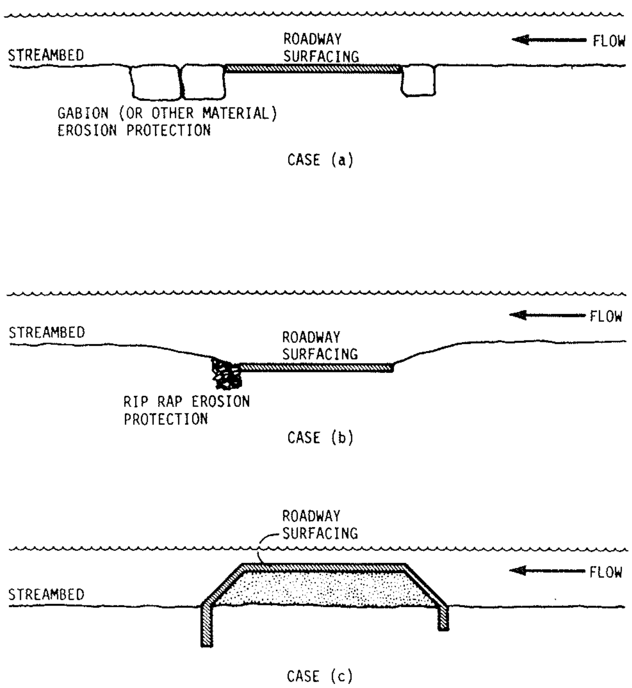

FIGURE 11 Typical fords—roadway cross section.

Application of a Low Water Stream Crossing

In a survey of LWSC use in the United States, 61 percent of the respondents reported they were used only on unpaved roads (*4*). Because paved highways have a geometric design and traffic control that are conducive to higher speeds, drivers' expectations are not consistent with the vertical profile encountered at LWSCs. In addition, because unpaved roads are limited to low traffic volumes, the use of LWSCs on these roads would involve a lower exposure to traffic. Carstens and Woo do not recommend the use of LWSCs on paved roads in Iowa.

The use of an LWSC design is based on an acceptance of periodic flooding. If flooding would isolate a place of human habitation, either an alternate design should be considered or an alternate emergency access route should be developed.

Approach Signing

The signing recommendations shown in Figure 12 are based on Carstens' and Woo's research (*4*). The recommendations were subsequently adopted by the Iowa DOT as recommended practice. According to Carstens and Woo, the intent of the regulatory sign DO NOT ENTER WHEN FLOODED is to preclude travel across the LWSC when the roadway is covered with water (*4*). Such a regulatory sign requires a resolution by the Board of Supervisors. The adoption of this sign in effect significantly reduces the applicability of an unvented ford.

Supplemental Signing

If the location of an LWSC is not apparent from a point approximately 1,000 ft in advance of the crossing, a supplementary distance plate may be used. The message "700 feet" would be displayed with the FLOOD AREA AHEAD sign. The sign would be 24 in × 18 in with a black legend on a yellow background.

An advisory speed plate may be used if the maximum recommended speed at the LWSC is less than the speed limit in effect, which is usually the case. The advisory speed plate would be installed in conjunction with the FLOOD AREA AHEAD sign. However, if a supplemental distance plate is used, the advisory speed plate would be installed in conjunction with the IMPASSABLE DURING HIGH WATER sign.

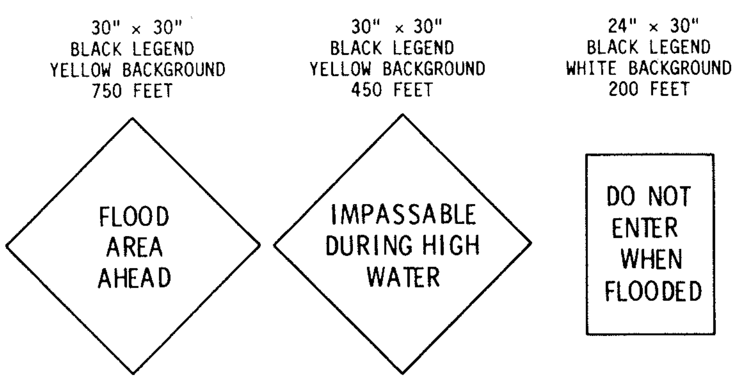

FIGURE 12 Signs recommended for installation at low water stream crossings.

Controls at a Low Water Stream Crossing

Various controls have been used to delineate the edges of the traveled way at an LWSC. Curbs are generally unacceptable because the flow of water tends to deposit mud and debris on the roadway. Attempts have been made at a few locations to create a series of small, raised curb blocks with tapered upstream slopes to provide for a smooth laminar flow. The use of any projections above the normal roadway surface will have an adverse effect on the self-cleaning aspect of the smooth cross-section. However, observations of existing applications, or further research in this area, are needed.

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Guidelines for the Design of Low-Cost Water Crossings

LOUIS BERGER, JACOB GREENSTEIN, AND JULIO ARRIETA

In Ecuador, as in many Third World countries, low-volume rural roads can only be economically justified when very low-cost bridges and simple water crossings (fords) are used. Traffic analyses indicate that in most cases the trucks that travel these roads carry loads that weigh less than 6 to 10 metric tons. Therefore, most of the drainage structures are designed to carry only 10 tons on two-axle light vehicles. Roads are designed according to AASHTO HS-15 standard loading in those locations where heavy traffic is generated from timber production or banana plantations. The standard AASHTO HS-20 live load cannot be economically justified for these low-volume roads. The traffic volume in rural regions is very low, which enables such economical structures as graveled fords to be used, and, when economically feasible, one-lane bridges with either complete or split decks. The relationships between the type of material, the span or length of the superstructure, and the cost are analyzed. It is primarily concluded that simple timber bridges made of stringers and transverse laminated decks are the most economical solutions for simple spans up to 17, 14, and 10 m for 6-, 10-, and 24.5-ton truckloads, respectively. Simple-span, split-deck, reinforced-concrete superstructures are feasible for spans of up to 30 m. Spans can be as long as 45 m if prestressed girders are used. Suspension bridges with timber decks and timber-stiffening trusses were built to carry 6-ton trucks or cattle wagons and were more cost-effective than timber or concrete structures. It was concluded that with the judicious reduction of the design standards of live loads, cross-sections, geometry, material specifications, and hydrologic and hydraulic considerations, construction costs could be reduced by 50 percent or more. These savings make it possible to justify the construction of many low-volume rural roads that would otherwise be impossible to finance.

Low-volume roads are needed in such developing countries as Ecuador and Colombia to provide access in agricultural and rural regions (*1*, *2*). A socioeconomic analysis is performed to determine which type of road is the most economical to build. The use of this methodology enables the least-cost road to be determined for any given traffic projection, degree of agricultural productivity, and extent and type of social and population activities.

Several types of low-cost rural roads exist in Ecuador: (a) earth or dirt roads that are 2.5 to 4.0 m wide and provide access only during the dry season, (b) 4.0- to 6.0-m-wide compacted subgrade or gravel roads, (c) 4.0- to 5.0-m-wide stone roads constructed mainly in the Andes region, and (d) 6.0- to 7.2-m-wide base course roads with or without blacktop. Construction of most of these low-volume roads can be economically justified only if the construction cost is minimized to achieve a feasible rate of return on the investment. The minimum initial rate of return required to justify investment in the construction of low-volume roads in Ecuador in 1984 to 1985 was 12 percent. This objective can be achieved only if low-cost water crossings are used to provide access.

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The economic horizon or lifetime of a low-cost road in Ecuador is 17 years. It was concluded in a study financed by the World Bank that all agricultural and other economic benefits could be achieved during this 17-year period, and the investment therefore would be justified (*2*). The minimization of costs and maximization of benefits during the economic horizon are both needed to optimize and justify road and bridge construction. Cost savings can be obtained by setting appropriate standards for certain design elements, such as design load, cross-sections, low-cost materials, and hydrologic and hydraulic design criteria, even though these criteria may appear to be substandard to the developed world.

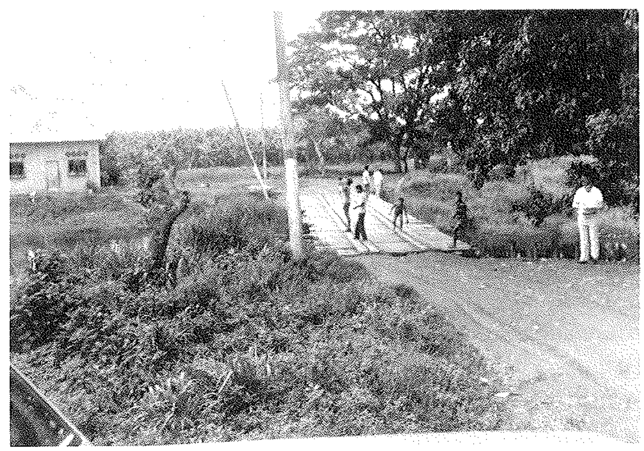
Typical low-cost bridges and water crossings in Ecuador are shown in Figures 1 to 5. A one-lane timber bridge in Puerto Viejo, Ecuador, that was designed to carry only one vehicle at a time with a total weight of less than 6 tons is shown in Figures 1a and 1b. A one-lane timber deck in Puerto Bartelo, Ecuador, that was designed to support a truck carrying less than 10 tons is shown in Figures 2a and 2b. A typical one-lane concrete and steel split deck that can carry one truckload of less than 10 tons is shown in Figures 3a and 3b. A one-lane concrete bridge that was designed to carry HS-15-44 trucks with a total weight of 24.5 tons is shown in Figures 4a and 4b. A ford-type water crossing is shown in Figures 5a and 5b. This type of ford is very common in the Ecuadoran Andes.

Basic design guidelines, typical cross-sections, and cost comparisons of low-cost bridges and water crossings in Ecuador are presented.

LIVE LOAD DESIGN

It is well-known that the transport of goods in the Third World is mainly performed by the private sector, which saves money by overloading its trucks. Recent projects in Ecuador and Colombia financed by the World Bank indicate that little is currently being done on the main roads to control truck overloading. This conclusion also appears to apply to other countries. As a result of this evidence, the structural division in the Ecuadoran road authority designed all bridges for both main and rural roads according to the AASHTO HS-20 standard truck loading. The lower AASHTO standard HS-15

(a)



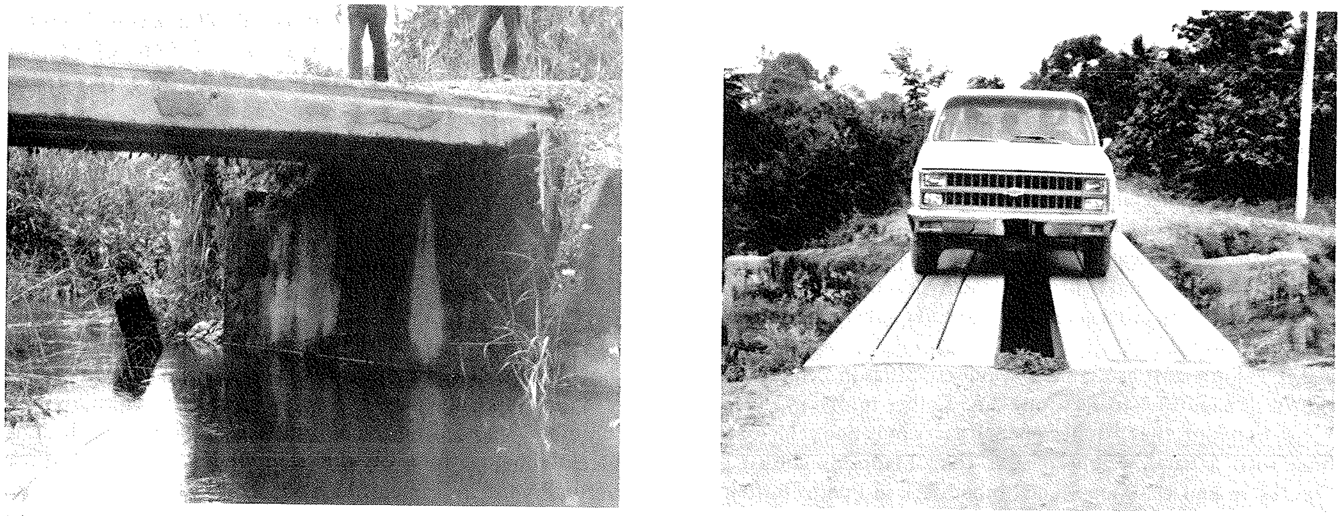
(b)

FIGURE 1 Two views of a timber bridge.

(a)

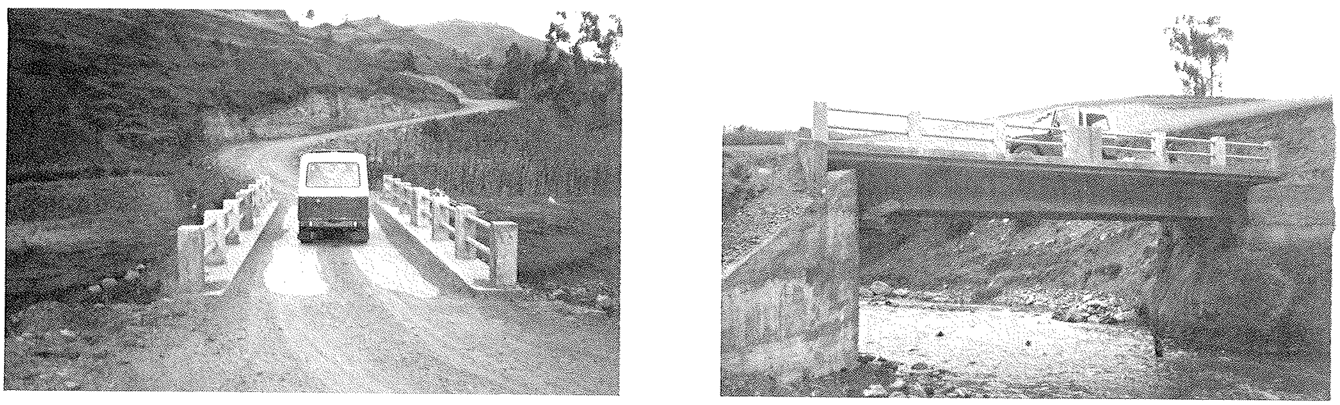
(b)

FIGURE 2 Two views of a timber deck.



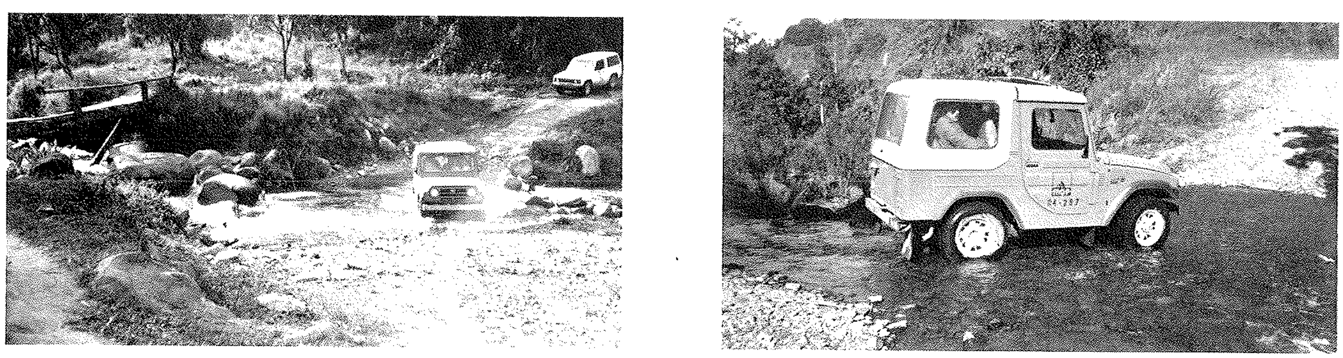
(a) (b)

FIGURE 3 Two views of a one-lane, split-beam deck.



(a) (b)

FIGURE 4 Two views of a one-lane concrete bridge design for HS 15-44.



(a) (b)

FIGURE 5 Two views of a low-cost ford in the Ecuadoran mountainous zone.

truck was only occasionally used in the design of rural bridges. Recent economic and transport studies in Ecuador indicate that the actual vehicle loading on rural roads is significantly lower (*1-3*). The largest vehicle on over 90 percent of the roads is a two-axle truck with a total weight of less than 10 metric tons. About 75 percent of the vehicles are pick-up trucks and light buses and trucks, with a total weight of 2 to 6 tons. An economic and traffic projection analysis indicated that the volume of traffic might increase slightly in most of the existing Ecuadoran rural roads, but no changes in vehicle type or total weight are expected (*1, 2*). In other words, the projected demand and economic growth, and the low standard of the road and pavement, make the use of oversized or overloaded vehicles infeasible (*1, 4*). Only in a few rural locations—regions with heavy traffic from timber-producing regions or banana plantations—can a AASHTO standard HS-15 live load be economically justified. These relatively few roads usually have higher design standards; a 6.0- to 7.2-m-wide base course pavement with or without blacktop is usually used. Based on these economic and traffic forecast analyses, the Ecuadoran road authorities decided that it was practical and economical to adopt lower design standards for live loads on most of the low-cost rural bridges. The following three load categories were adopted (see Figure 6):

- An M6 truck with 1,200 kg and 4,800 kg on the front and rear axles, respectively;
- An M10 truck with 2,000 kg and 8,000 kg on the front and rear axles, respectively; and
- An HS-15 load with 2,720 kg on the front axle and 10,880 kg on each of the two rear axles, for a total of 24,480 kg.

The AASHTO standard HS-20 live load was not found to be economically justified for these low-cost roads.

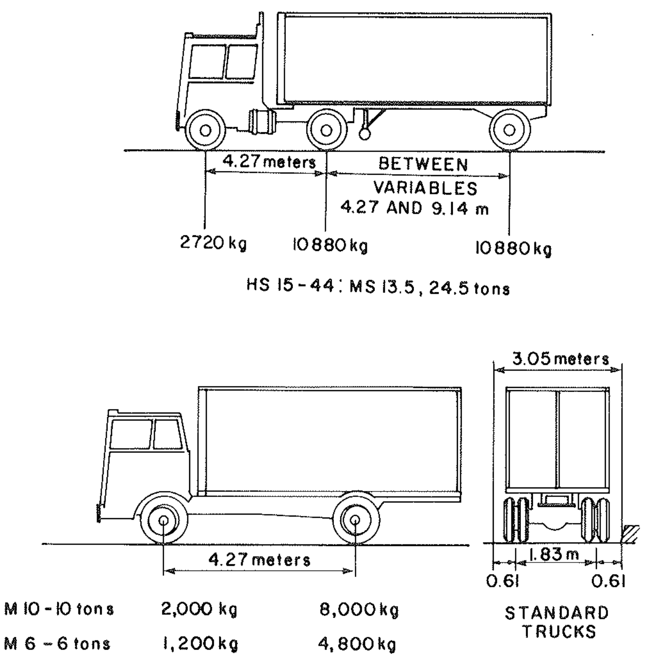


FIGURE 6 Two designs for live loads on rural bridges.

HYDRAULICS AND HYDROLOGY

The deforestation that occurred in Ecuador the last few years has resulted in severe flooding in rural regions. The deforestation also caused an increase in the flooding discharge and a reduction in its duration. The deforestation and changes in flooding characteristics occurred more often in the rural or remote regions and less often in the vicinity and area of influence of major highways. Previous hydraulic records for rural bridges therefore should be analyzed with caution to prevent rural bridges from failing during floods, as shown in Figures 7a and 7b. A typical scour failure in a rural road bridge in the Ecuadoran province of El Oro is shown in Figure 7a. The scouring caused 1 ft of settlement in the center abutment. The bridge is still used for light and partially loaded trucks.

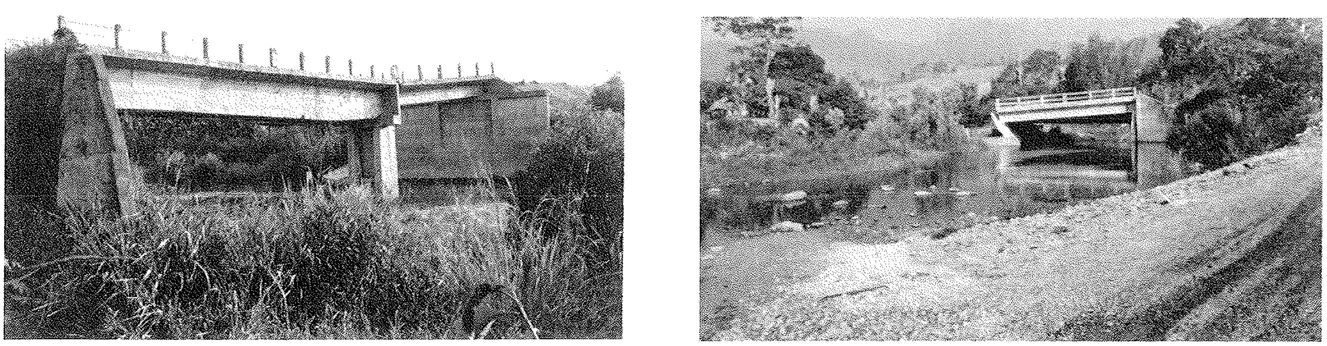
A typical total bridge failure that was caused by flooding in 1982 is shown in Figure 7b. Although the hydraulic analysis should be precisely executed to eliminate any unexpected failures, especially in cases in which changes have recently occurred in the flooding pattern, the hydraulic design criteria should permit bridge construction costs to be minimized. The criteria for main roads specify or require that the clearance between the bridge's bottom deck and the maximum flood level should be 1 ft (30 cm) for the occurrence of a storm every 100 years. In rural road design, the storm period can be reduced to 25 years. This period is approximately equal to 150 percent of a road's economic lifetime. Experience also indicates that the water clearance should remain at 1 ft unless the water velocity is slow and accumulation of debris is not expected. A slow stream is defined as one in which the slope is less than 0.5 percent and the maximum velocity is below 10 ft/sec.

LOW-COST BRIDGES

Concrete Bridges

Before 1984, the Ecuadoran rural bridges were designed according to AASHTO standards (5). A cross-section of a typical two-lane bridge is shown in Figure 8. Such bridges were designed in 1980 to carry an AASHTO standard HS-20 truck. Because two-lane bridges were found to be economically infeasible for rural roads, a new standard was established (1, 2).

A one-lane bridge (Figures 9a and 9b) was established as the highest standard that could be economically justified for rural



(a) (b)

FIGURE 7 Two views showing typical bridge flood failure in the province of El Oro.

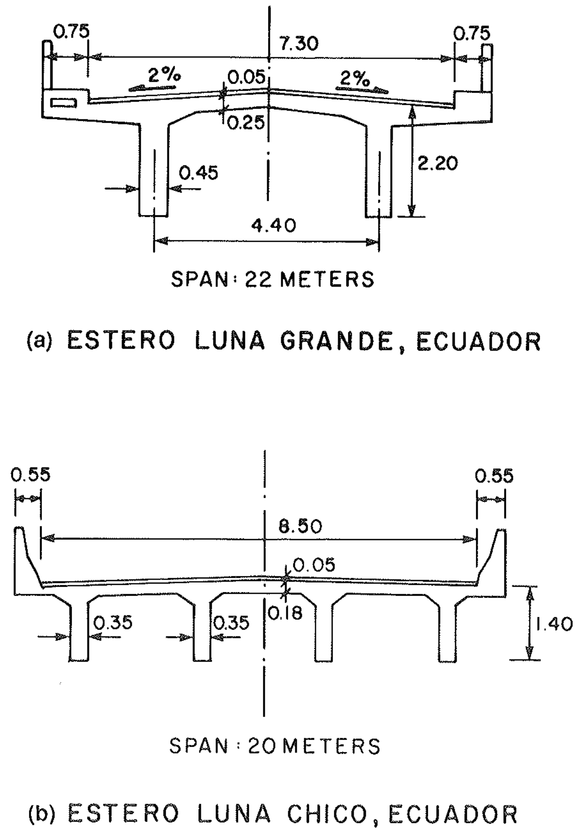


FIGURE 8 Typical two-lane rural bridge design for AASHTO HS-20 truck loading in Ecuador.

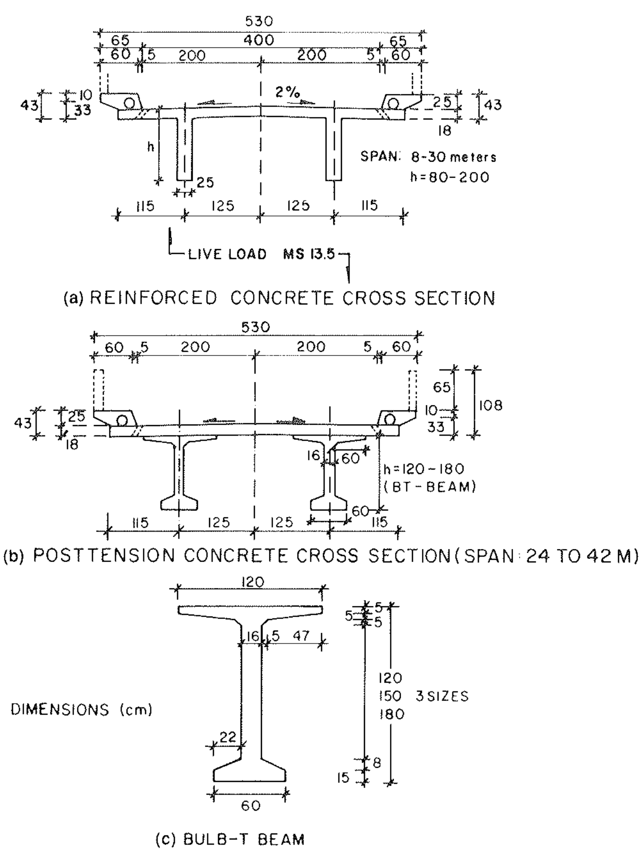


FIGURE 9 (a) One-lane reinforced concrete bridge cross-section; (b) one-lane post-tensioned concrete cross-section.

roads. The bridge cross-sections shown in Figures 9a and 9b are designed to carry an AASHTO standard HS-15 truck. A reinforced concrete cross-section that is usually feasible for a span of 8 to 30 m is shown in Figure 9a. A post-tensioned concrete cross-section that is usually feasible for spans between 25 and 45 m in length is shown in Figure 9b. The sidewalk and guardrail shown in Figure 9b are provided when pedestrians, cattle, and vehicles are to use the bridge. Cattle can use the bridge only if it crosses deep water.

An economical one-lane, split-deck bridge is shown in Figures 10a, 10b, and 10c. The split cross-sections shown in these figures are used for 6- and 10-t trucks (M6 and M10). A simple, multibeam, precast concrete bridge is used for short spans of usually 8 m or less. The construction procedure is very simple. The slabs are precast on the river bank and are easy to place and tie together to form the split deck. A typical cast-in-place split-deck bridge that is primarily used for bridge spans of 8 to 24 m is shown in Figure 10b. Cast-in-place, reinforced concrete bridges are popular in developing countries, such as Ecuador, Peru, and Colombia. This technique is also well-known to local industry. In addition, the only materials that have to be transported to the site are the reinforcing steel and cement.

The simple, split-lane reinforced bridge shown in Figure 10b is light and safe to use by vehicles and pedestrians. It is worthwhile to mention that the geometry of the split lane with the inside curb shown in Figures 10a and 10b contributes to its traveling safety. The split lane contributes naturally to speed

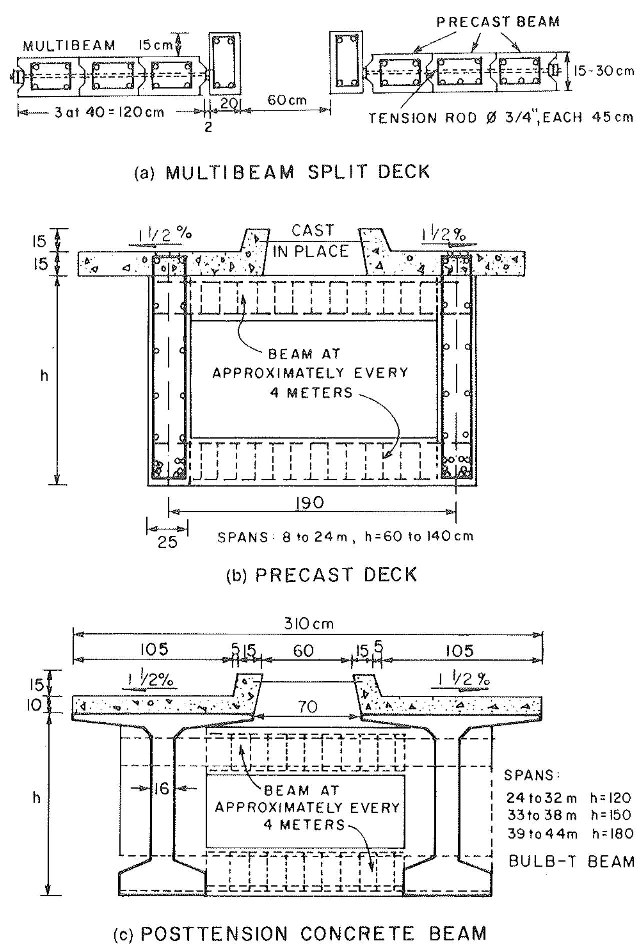


FIGURE 10 Typical split deck for low-cost bridges.

reduction and the inside curb is effective in preventing vehicles from sliding off the bridge. A split deck with prestressed girders of the sort shown in Figure 10c is more feasible in Ecuador when the single span exceeds 24 m.

In developing countries, such as Ecuador, Colombia, and Peru, the required pretensioning equipment is usually not available at a reasonable distance from the site; therefore, most of the prestressed bridge elements are post-tensioned. Standard post-tensioned girders and cast-in-place slabs that are practical for low-cost bridges with spans ranging between 25 and 45 m are shown in Figure 10c. Long, low-cost bridges are rarely constructed with a single span of over 45 m.

Each of these cases has been studied in detail in regard to the live load and bridge element type. The live load of bridges on roads classified as low-volume and low-cost in Ecuador consists of cattle or a maximum truckload of 6 tons.

Timber Bridges

In the past few years timber has played an important economic role in the construction of low-cost bridges in countries that comprise the Pacto Andino, which include Colombia, Venezuela, Ecuador, Peru, and Bolivia. These countries are rich in natural resources, especially in their huge tropical and subtropical zones. New technology in regard to timber structural elements is provided through the Agreement of Cartagena-Colombia. This agreement provides assistance and technology for the classification of timber, improved mechanical properties, pest control, and processing and treatment of timber (*6*). Further information on the design and use of timber bridges is available elsewhere (*7-9*).

It was determined that the currently unlimited source of natural raw materials, and the availability of technical assistance in production, make timber a feasible and practical alternative for bridge construction (*6*). Two other factors contribute to the feasibility of using timber for rural bridge construction. First, timber elements, especially in decks, are light; they weigh approximately one-third to one-fourth of the weight of an equivalent concrete deck. The use of timber therefore reduces foundation costs. Second, the economic life of a rural road is only 15 to 20 yrs; therefore, the investment in lumber treatment can and should be limited to this period only.

A simple Ecuadoran timber bridge with a laminated deck is shown in Figures 11a and 11b (*3*). The cross-sections are made of timber stringers that span between abutments or piers and

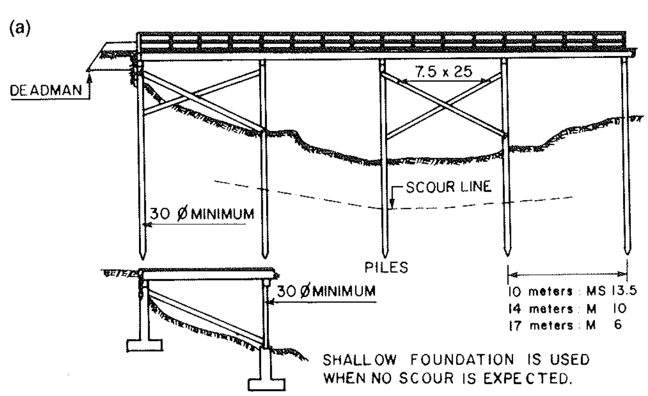


FIGURE 11 Typical timber bridge with laminated deck.

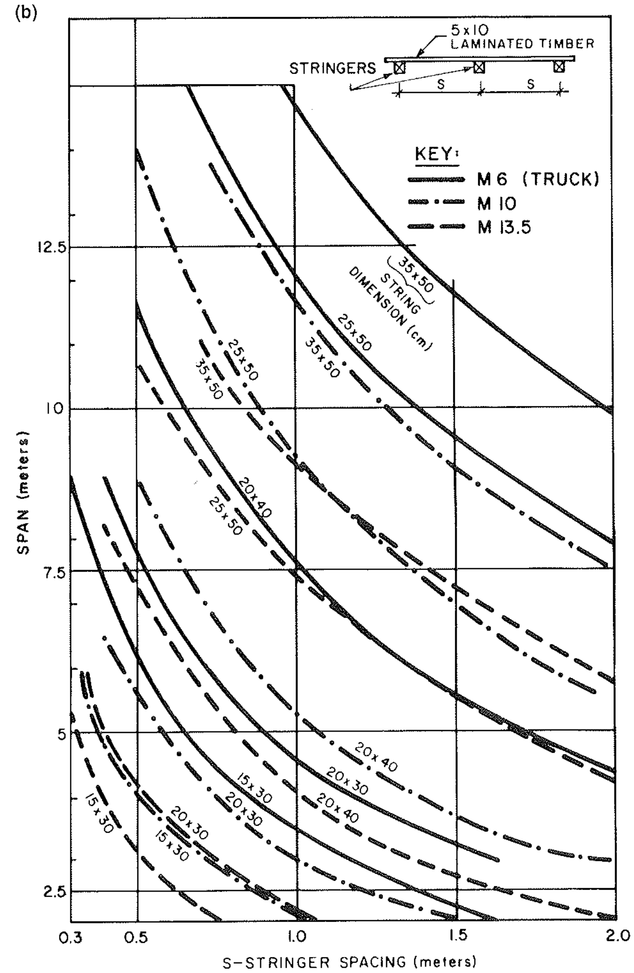


FIGURE 11 continued

transverse laminates that are nailed to one another and to the stringers. The transverse laminates are sometimes spanned over trussed or solid-web girders. The timber deck cross-section shown in Figure 11b was designed to a maximum span length of approximately 17, 14, and 10.5 m for truckloads of 6, 10, and 24.5 tons, respectively. This structure is economical, quick and easy to construct, and easy to maintain.

The design guidelines for the dimensions and optimum location or separation of the stringers to carry these traffic loads are shown in Figure 11b (*3*). For example, for a span of 10-m, relatively heavy stringers 25 cm wide and 50 cm high should each be spaced at 1.35, 0.85, and 0.60 m to carry live loads of 6, 10, and 24.5 tons, respectively. The maximum and most economical timber deck span can be increased by approximately 40 percent by using longitudinal or transverse cable post-tensioning. The implementation of this technique is still very limited in the developing countries of South America.

Another type of low-cost timber bridge was developed and implemented by the United Nations Organization for Industrial Development (UNOID). This prefabricated timber bridge consists of triangular moduli 3 m long that are joined together at the site. These short elements are easy to transport. Like the other timber bridges that were previously described, the UNOID bridge can be assembled easily by unskilled labor.

A typical cross-section of a UNOID bridge is shown in Figure 12. This one-lane timber bridge is now promoted in

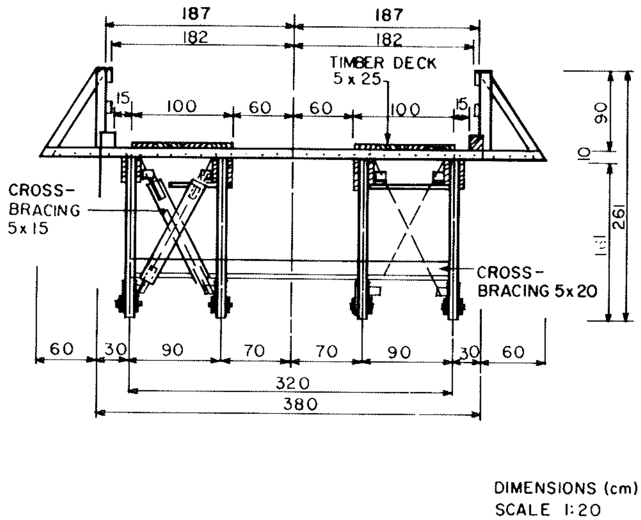


FIGURE 12 ONOID timber bridge.

Africa, Central America, and Ecuador by the United Nations and is designed to carry 6-, 10-, and 24.5-t trucks in a single span of 24, 21, and 15 m, respectively. As was mentioned previously, a long, single-span low-cost bridge is only occasionally needed for both vehicles and cattle. This special need primarily exists in the eastern tropical Ecuadoran Amazonas region. A typical bridge is shown in Figure 13. This bridge is designed only for light vehicles of 6 t or less, or for passing cattle. The spans of this single bridge vary between 45 and 120 m; the only nontimber elements are the reinforced concrete towers, the anchor blocks, and the cable.

Other Low-Cost Water Crossings

In many cases in which international or government funds are limited and timber for split-deck bridges is unavailable, other

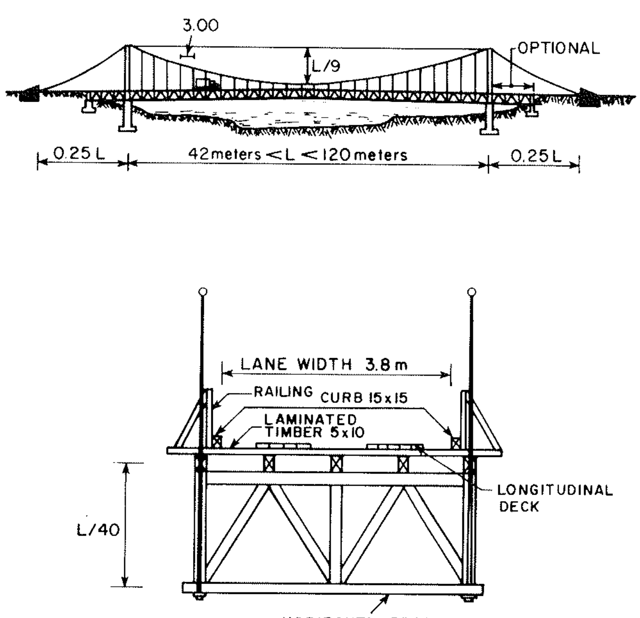


FIGURE 13 Single-span bridge with timber deck and timber-stiffening trusses.

low-cost water crossings can be built. In most of these cases, good judgment and experience with locally available materials can be used instead of standard specifications and structural analysis. The most commonly used and economical types of water crossings in Ecuadoran rural regions are described in the following paragraphs. All three types provide reasonable access for 3 to 5 yrs with no major repairs.

One type of low water crossing consists of beams of solid webs that are nailed to two flanges made of two layers of boards that support a split in the timber deck. This structure is used in spans of up to 25 m and mainly carries vehicle loads of less than 6 tons. This structure can deteriorate rapidly if moisture accumulates between the boards.

A no-stringers deck is designed for span lengths of up to 12 m. The bridge is made of timber laminates that run parallel to its longitudinal axis. These laminates are the only element in the superstructure.

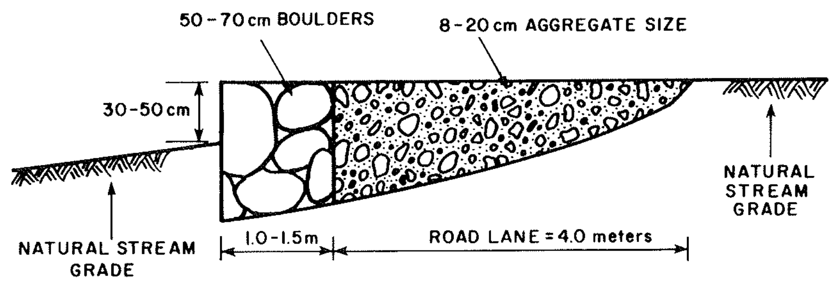
Graveled fords are commonly used in the mountainous regions of Ecuador (Figure 5). Fords are used as low-cost water crossings on almost every unpaved rural road in this region. A typical cross-section of a ford is shown in Figure 14 for both steep and flat water crossings. The construction of this type of crossing is usually labor-intensive. The surface of graveled forms usually performs adequately for 3 to 5 yrs in the Ecuadoran Andes. Maintenance is rather simple; it is also performed by manual labor with a relatively minor cost. Experience in Ecuador clearly indicates that the construction and maintenance costs of a ford are always less than a fraction of those for a single-lane, low-cost bridge. They cost approximately 50 to 100 U.S. dollars/linear meter when local materials are available.

COST COMPARISON

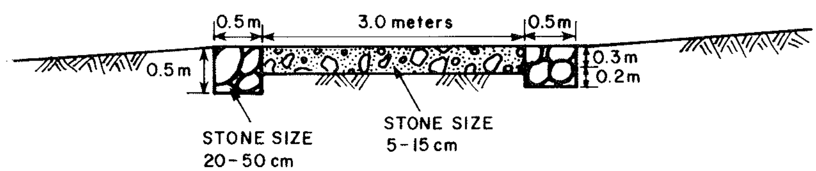
The construction costs of rural concrete bridges that are designed to carry AASHTO standard HS-20 trucks are given in Table 1, in which the costs of the bridge deck and the bridge as a whole are broken down. As shown in Table 1, the total construction cost of a two-lane, 8.5-m-wide concrete bridge varies between 1,600 and 2,200 U.S. dollars/linear meter (1986 prices). The cross-section of this typical bridge is shown in Figure 8.

Significant savings in cost were achieved in Ecuador by using the low-cost bridges described in this paper. The savings in cost are given in Table 2, in which the relationships between the total bridge construction cost per linear meter, traffic loading, and type of bridge are shown. It is clearly indicated in Table 2 that the use of timber bridges significantly reduces the cost of construction. The average total cost of a one-lane timber bridge in Ecuador in 1984 and 1985 was approximately 400, 500, and 650 U.S. dollars/linear meter (1986 prices) for truckloads of 6, 10, and 24.5 tons (MS 13.5), respectively. This cost is approximately 20 to 30 percent of that of the two-lane, standard concrete bridge that was previously used in Ecuadoran rural regions (Figure 8b).

The concrete split-deck bridge is another economical water crossing. The average total construction cost of the multibeam bridge shown in Figure 10a varies between 450 and 750 U.S. dollars/linear meter, which is approximately 30 to 45 percent of that of a two-lane standard bridge. The higher cost values are related to heavier truck loads. This bridge type is recommended



(a) STEEP SLOPE WATER CROSSING



(b) FLAT SLOPE WATER CROSSING

FIGURE 14 Simple ford cross-section (Ecuadoran mountainous zone).

TABLE 1 CONSTRUCTION COSTS OF RURAL BRIDGES IN ECUADOR IN 1984

Bridge width	Element	Economic cost x 10^3 (U.S. $/linear meter)
6.0 meters	Deck only	0.6 - 0.7
	Entire bridge	1.2 - 1.6
8.5 meters	Deck only	0.8 - 1.0
(Fig. 8.b)	Entire bridge	1.6 - 2.2

for short spans of about 8 m. The construction cost of the concrete split-deck bridge shown in Figures 10b and 10c usually varies between 400 and 900 U.S. dollars/linear meter for a truckload of 10 tons. This cost is approximately 28 to 40 percent of that of a standard two-lane bridge. The prestressed split-deck bridge is more economical when the single span of the bridge is over about 30 to 45 m.

Additional cost savings can be obtained by constructing a ford, as shown in Figures 5 and 14. The average construction cost of a one-lane ford was 50 to 120 U.S. dollars/linear meter. The variation in cost reflects the availability and cost of local materials and skilled labor in the vicinity of the project.

SUMMARY AND CONCLUSIONS

Investment in low-volume roads in developing countries can be economically justified only when very low-cost bridges and simple water crossings are used. Cost savings can be obtained by setting appropriate standards for the design elements of load, cross-sections, low-cost materials, and hydrologic and hydraulic design criteria, even though these criteria may appear to be substandard in the developed world. The following conclusions can be made.

Economic projection and traffic analysis indicate that truckloads of less than 6 to 10 metric tons are usually traveling along these roads. Therefore, most of the drainage structures are designed to carry 6 to 10 tons on two-axle, light vehicles. The AASHTO HS-15 or MS 13.5 loadings are used in the design of these bridges only when heavy traffic is expected from timber-producing regions or banana plantations. The AASHTO standard HS-20 live load cannot be economically justified for these low-volume roads.

The economic lifetime of a rural road in Ecuador was determined in 1984 and 1985 to be 17 years. All agricultural and other benefits can be achieved during this 17-year period. The investment in construction and maintenance costs can therefore be justified.

The traffic volume in Ecuadoran rural regions is often less than 100 vehicles per day; in fact, it is usually less than 20 to 50 vehicles per day. These low traffic volumes enable the use of graveled fords, such as those shown in Figures 5 and 14. The average construction cost of a one-lane ford was 50 to 120 U.S. dollars/linear meter. These timber bridges were found to be the most practical and economically justifiable bridges for simple spans of up to 17, 14, and 10 m, with 6-, 10-, and 24.5-ton truckloads, respectively.

The recommended storm period in the design of bridges on rural roads is 25 years. The recommended clearance between the maximum storm water level and the bridge should be 1 ft, unless the water velocity is very low. In cases in which the stream's ground slope is less than 0.5 percent, the maximum velocity is less than 10 ft/sec, and no accumulation of debris is expected, the water clearance could be reduced from 1.0 ft to 0.5 ft.

TABLE 2 COST COMPARISON OF LOW-COST BRIDGES (U.S. $1,000/linear meter)

Bridge type and loading	Bridge length (meters) 8 - 10	15	20 - 21	30	39 - 40
Timber bridge (Fig. 11)					
Load					
M6	.35-.45	.35-.45	.30-.40	.40-.55	.45-.60
M10	.40-.51	.43-.59	.45-.65	.45-.65	.54-.75
MS 13.5 (HS - 15)	.45-.62	.56-.77	.56-.77	.56-.80	.65-.90
Timber bridge (UNOID; Fig. 12)					
Load: MS 13.5	.62-.86	.46-.64	.43-.59	.48-.66	.46-.64
Multibeam (Fig. 10.a)					
Load					
M6	.40-.55	–	–	–	–
M10	.50-.68	–	–	–	–
M13.5	.62-.86	–	–	–	–
One - lane reinforced precast concrete split deck (Fig. 10b)					
Load: M10	.40-.60	.40-.60	.48-.66	.53-.73	.70-.97
Full-width deck (Fig. 9a)					
Load					
M10	.50-.68	.54-.75	.61-.84	.67-.92	.88-1.21
MS 13.5 (HS - 15)	.63-.86	.72-.99	.77-1.10	.85-1.17	.93-1.28
One - lane prestressed split deck (Fig. 10c)					
Load: M10	–	–	–	.60-.84	.60-.88
Full - width deck (Fig. 9b)					
Load: M13.5 (HS - 15)	–	–	–	.80-1.10	1.04-1.43

Timber bridges made of stringers and laminated decks (Figure 11) appear to be an economical and practical solution for low-cost water crossings. The total construction cost per linear meter of one-lane timber bridges in Ecuador in 1984 and 1985 was as follows:

Load	*$ U.S.*
6-ton truckloads (M6)	350 to 450
10-ton truckloads (M10)	400 to 650
24.5-ton truckloads (MS13.5)	450 to 750

These costs are in the range of 20 to 40 percent of an AASHTO standard, two-lane bridge design that was previously used in the rural regions.

One-lane, split-deck, reinforced concrete bridges (Figures 9 and 10) obviously have a longer life expectancy than timber bridges and they can still be considered as adequate, economical alternatives. The most practical alternatives are multibeam, reinforced, and prestressed beam decks, as shown in Figures 10a, 10b, and 10c, respectively. The average total costs of these bridges are about 450 to 800 U.S. dollars/linear meter. The split-deck, reinforced concrete simple span has been found to be the most practical and economically justifiable bridge type for simple spans of up to 30 m. Spans can reach 45 m when prestressed girders are used.

The total construction cost of a full-width, one-lane, precast concrete bridge (Figure 9a) is approximately 20 to 30 percent more expensive than the equivalent split-deck bridge shown in Figure 10. It should be noted that the full one-lane prestressed bridge is always designed to carry HS-15 truckloads, whereas the split-deck bridge is designed for a standard 10-ton truck (M10).

It can be concluded that with the judicious reduction of design standards of live loads, cross-sections, geometry, material specifications, and hydrologic and hydraulic considerations, the construction costs of water crossings can be significantly reduced. This also means that it is economically feasible to make improvements to low-volume rural roads.

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Use of Concrete Median (Jersey) Barriers as Ford Walls in Low Water Crossings

RODNEY F. MENDENHALL AND JOHN R. BARKSDALE

The use of precast concrete median (Jersey) barriers as ford walls on low-volume roads is described. Ford walls are used on U.S. Forest Service roads to stabilize low water stream crossings. This is an acceptable practice on roads that have been temporarily closed for 1 or 2 hours as a result of flooding from sudden and intense storm runoff. The barriers are readily available, precast units that can be transported to the site and installed with conventional equipment that is used to maintain low-volume roads. Modified barriers with steel caps have also been used successfully to prevent erosion of the top of the concrete wall as a result of abrasive bedload movement during high water flows. Ford walls that were constructed with concrete median barriers have been used on hundreds of low water crossings in the desert and mountainous regions of the southwestern United States. These barriers have proved to be an efficient, low-cost alternative to conventional, cast-in-place concrete walls.

The U.S. Department of Agriculture, Forest Service, manages a network of approximately 300,000 mi of road on almost 200,000 million acres of land. These roads are needed to manage a variety of resources and activities, such as timber harvest, recreation, mining, forage, fire protection, and other forest-related activities.

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The Southwestern Region of the Forest Service manages approximately 46,500 mi of road in the states of Texas, Oklahoma, New Mexico, and Arizona. About 70 percent of these roads are of a low standard; most are single-lane roads with dirt or pit-run surfacing and an average daily traffic count of less than 50.

The terrain in the Southwest varies from low Sonoran desert to mountains that are more than 10,000 ft high. Annual rainfall varies from 5 in in the desert to 50 in in the upper pine and alpine forests. Sudden, intense rainstorms are common during the rainy season in July and August. Stream beds that are normally dry become raging torrents within a matter of minutes. These storms often cause injuries and occasionally cause deaths.

A wide range of soil types exists in the Southwest. Much of the soil is composed of highly erodable sandy clays, decomposed granite, and plastic clays. Many roads are impassable for short times during storm runoff because the crossings are flooded and road surfaces are soft and slippery. Good drainage is the key to reducing repair and maintenance expenditures.

THE PROBLEM

The Southwestern Region of the Forest Service has experienced seven major storms since 1972 that have caused over $25 million in damage to roads. Much of the damage occurred at low water crossings. Because of budget restrictions and low traffic counts, drainage structures such as large culverts and bridges are the exception instead of the rule.

The ford structures at low water crossings typically consist of cast-in-place ford walls that are placed in the downstream shoulder of the road to stabilize the road grade at the crossing. These walls are approximately 3 ft high and 6 in thick and may have a footing. Most of the structures were constructed by miners, ranchers, loggers, and road maintenance crews, often with substandard materials. Most of the structures were not designed in accordance with good engineering practices. The walls were subsequently damaged by the movement of rocks in the stream bed. The ends of the walls were also undermined and scoured as a result of undersizing.

Past repair methods involved extending or replacing the wall with a reinforced cast-in-place concrete wall. A 3/8-in-thick steel plate was attached to the tops of some walls to prevent bed load movement from eroding the concrete during floods. Quality concrete is available in this region, but haul distances of 50 mi or more are common. The excavation, forming, mixing, and curing of concrete and dewatering in live streams posed difficult construction problems.

AN ACCEPTABLE ALTERNATIVE

Forest Service engineers faced with tight time schedules for storm damage repairs studied alternate ways to reduce the costly repair and maintenance of ford crossings. The use of a system of precast median (Jersey) barriers as ford walls was a solution to this problem. These precast sections are widely used as temporary traffic separators during construction on state highways and are readily available in new and used condition (see Figure 1).

Jersey barriers are available in lengths of up to 20 ft. A standard section 12 ft, 6 in in length and about 5,000 lbs in weight was selected. This length can be easily transported and handled with equipment that is typically used in the construction and maintenance of low-volume roads.

Design

The structure of the ford wall consists of a series of precast concrete barriers that are embedded at a right angle to the stream and pinned and tied together with a length of 9/16-in cable and clamps (see Figure 2). The end sections are typically sloped upwardly to form a weir shape that forces the water to flow over the center of the structure. The center section is designed to be level.

A 1-ft-wide strip of 70 to 100 equivalent opening size (EOS) geotextile fabric is placed over each joint. The fabric will therefore retain fine-grained material on the upstream side of the wall and prevent holes from developing in the road grade.

The drainage structure should be sized to pass the anticipated storm runoff. Forest Service engineers perform a hydrological analysis for 50- and 100-yr stream flows for major structures. Structures of this type are sized for a 25-yr flow by using an appropriate weir formula or Manning's equation.

The structure is protected with riprap or gabions downstream, as required (see Figure 1). This process is described in many other publications. A 3/8-in-thick steel cap is placed on the structure to prevent bed load movement (boulders) from eroding the top of the concrete. This is easily done at the casting yard because the structures are cast upside down and a C6 × 10.5 steel channel is laid in the bottom of the form and anchored to the concrete.

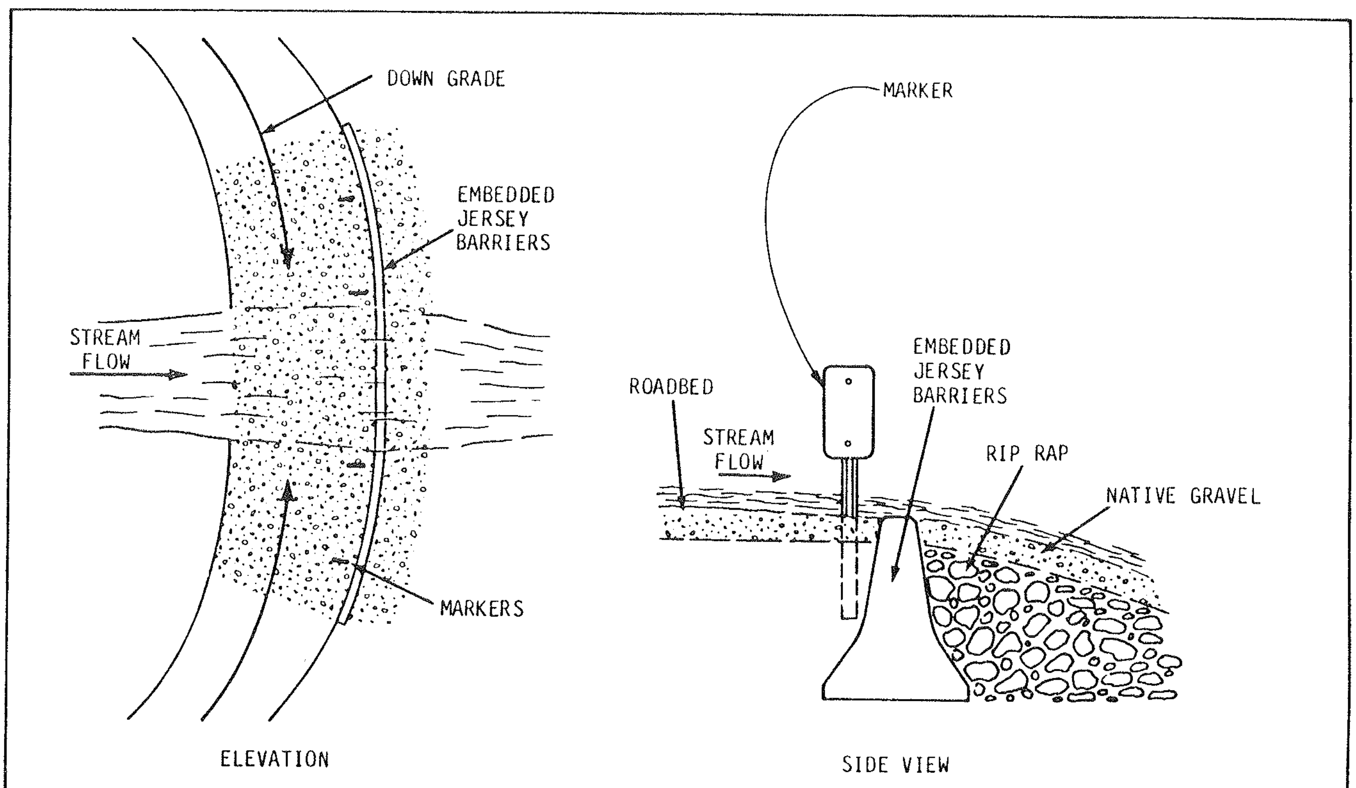

FIGURE 1 Typical installation of barriers.

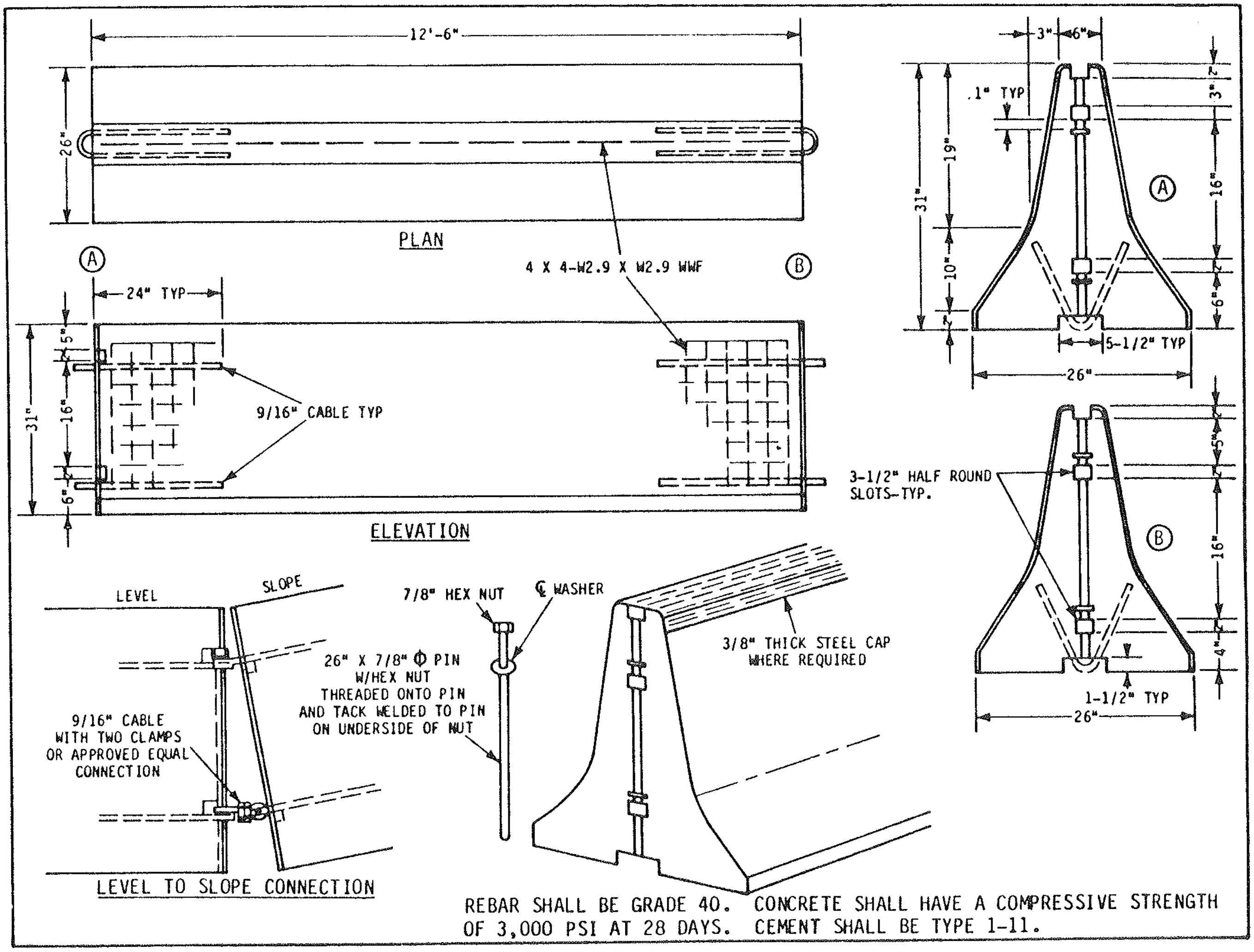

FIGURE 2 Typical fabrication details.

Installation

The equipment needed to install the structures consists of a rubber-tired backhoe and a boom truck or front-end loader large enough to handle the 5,000-lb load.

A trench is dug to the designed shape of the weir. The 12-ft, 6-in sections are then set directly into the trench and pinned or tied together with cable (see Figure 2).

CONCLUSIONS

The following benefits were realized by using precast concrete barriers as ford walls:

- They can be installed in intermittent or live streams without the complicated dewatering or diversion of water required for cast-in-place walls;
- The problem of transporting, forming, mixing, placing, and curing concrete at remote sites is eliminated;
- The foundation requirement is reduced because the base of the barrier is wide;
- Precast walls can usually be salvaged and reinstalled if they are washed out in extreme floods; and
- The purchase and installation costs are reduced from a range of $100 to $200 to $40 to $50/linear ft.

The use of precast concrete median barriers is recognized by the Forest Service as an economical, timely, and effective method of stabilizing ford crossings on low-volume roads.

Culverts Versus Dips in the Appalachian Region: A Performance-Based Decision-Making Guide

RONALD W. ECK AND PERRY J. MORGAN

Based on a literature review and field survey, specific factors that need to be considered in the decision to use culverts or broad-based dips for cross-drainage on low-volume roads were identified. Detailed roadway and environmental information was collected at 19 field sites in the Appalachian region to assess the performance of dips and 18-inch aluminum pipe culverts under a variety of conditions. Performance was rated as either acceptable or unacceptable by a survey team that made a field examination of the drainage structure. Overall, 227 culverts and 255 broad-based dips were assessed. Failure rates for culverts and dips were 7.5 percent and 27.5 percent, respectively. Distress types noted for the culverts rated as unacceptable were sloughing of the cut slope, clogging of pipes and inlets, and erosion of the fill slope. The most common distress types for the dips rated as unacceptable were erosion of the fill slope, rutting, siltation, and ponding. A number of specific conclusions regarding the design and location of dips and culverts were presented to document cases in which one device was more appropriate and cost-effective than another. A decision-making framework, in the form of a flowchart, was developed to assist engineers and foresters in selecting the appropriate drainage device for a particular application.

Drainage is one of the primary concerns in locating and designing low-standard roads that may serve only 0 to 50 vehicles per day (vpd). Drainage must always be adequate if a road is to remain usable. Several types of drainage devices are used to control water flow. Probably the most common type of device is the culvert, which is a closed conduit that carries surface water across or from the road right-of-way. Another device is the broad-based dip, which is a depressed out-sloped section of roadway that acts as a water catchment and drainage channel. Dips can be used instead of culverts for cross-drainage in locations in which no intermittent or permanent streams are present. The plan and profile of a typical broad-based dip are shown in Figure 1. The dip under discussion here should not be confused with low water stream crossings that are frequently found on paved low-volume roads.

Some controversy currently exists regarding the relative benefits and costs of dips and culverts. Some suggest that metal culverts are superior for most drainage needs (*1*, *2*). The initial cost of culverts is high compared with simple drainage devices, but culverts have long lifetimes, require relatively little maintenance, and are essentially unnoticed by road users.

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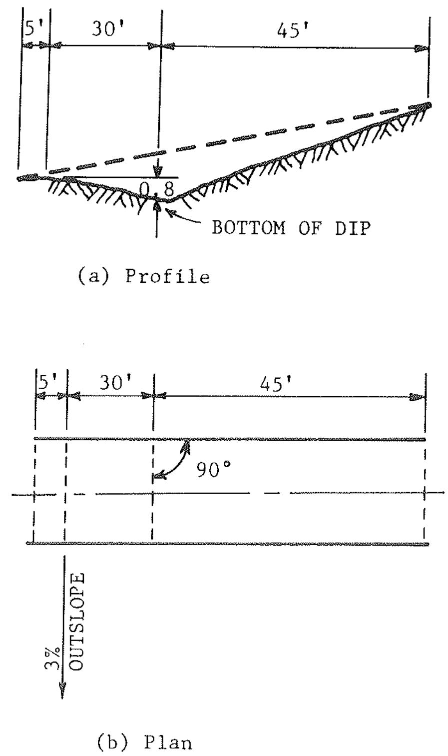

FIGURE 1 Plan and profile of a broad-based dip currently used by USFS national forests in North Carolina.

Others promote broad-based dips because they have several advantages (*3*, *4*). Dips have a relatively low initial cost, and unlike culverts, dips can be used without the expense of a ditch line. It has been reported that properly constructed dips have low maintenance costs and, like culverts, do not increase wear on vehicles or reduce hauling speeds (*3*, *5*). However, one disadvantage of broad-based dips is that equipment operators need special training to be able to construct dips properly. Therefore, dips are often not built according to intended specifications.

Design criteria have been established for both broad-based dips and culverts, although actual device dimensions and other details may vary from one geographic region to another. Most drainage devices, if constructed according to specifications and if placed at an appropriate location, will perform satisfactorily for many years. However, if the device is not built according to specifications or it is not properly located, serious problems can result.

An improperly placed or poorly constructed culvert could result in clogging of the pipe or erosion of the roadway or fill slope. Both of these situations can generate siltation and sedimentation, which would consequently degrade the forest vegetation and water quality. A clogged culvert not only increases the likelihood of the roadway washing out in the vicinity of the structure, but it may also introduce the possibility of damage to down-grade drainage structures.

A poorly constructed dip can result in a number of problems, including erosion, siltation, rutting, or ponding of the dip or roadway. Erosion necessitates actions to protect the fill slope and immediate down-grade streams, and replace lost material. Siltation calls for the removal of debris (mostly soil and rock particles) from the dip. Rutting and ponding often require that the dip be reconstructed, because these two types of distress create a build-up of mud or water that eventually creates an impassable roadway. If allowed to continue unabated, the economic costs associated with all of the problems just mentioned can be quite high.

Based on a limited field survey and discussions with forest road designers, builders, and users in the Appalachian region, it became apparent that whereas dips and culverts each have their place as a drainage device on low-standard roads, there are certain conditions in which one is more appropriate than the other. However, no formal engineering study had ever apparently been made of this issue. A need exists to objectively determine through the use of actual field data and an engineering economic analysis, under what conditions conventional metal culverts are more appropriate than broad-based dips on logging roads in the Appalachian region.

STUDY OBJECTIVES

A research project was undertaken to answer this question. Several specific objectives were established to address the overall goal:

- Based on a literature review and field survey, specific factors were identified that had to be considered in the decision to use culverts or dips.
- An experimental design was to be developed to collect detailed data at a number of field sites in the Appalachian region to assess the performance of dips and culverts under a variety of conditions. It should be noted that only aluminum culverts were considered in this study, because they were the only type of drainage device used in the Monongahela National Forest in West Virginia, where the field investigation was conducted.
- An economic analysis was to be conducted of 18-in aluminum culverts and broad-based dips, in which construction, maintenance, and road user costs were considered. The results of the economic analysis are presented elsewhere (*5*).
- Based on the field study and the economic analysis, specific conditions were to be recommended under which culverts or broad-based dips should be installed.

DATA COLLECTION

The factors involved in the performance of a drainage device must be known in order to develop guidelines to assist in the selection of the most appropriate type of cross-drainage. Based on a literature review, field survey, and discussions with practitioners, those factors that affected performance and that needed to be considered in the selection of a dip versus a culvert were identified. In order to establish a convenient framework for decision-making and to formulate the experimental design for the field study, the factors just mentioned were grouped into several major categories, as shown in Figure 2.

In order to determine specific situations in which one drainage structure was more appropriate than another, a field study was designed to evaluate the performance of a large number of broad-based dips and 18-in aluminum pipe culverts in the Monongahela National Forest. The experimental plan was formulated in such a manner that the effects of the variables that influenced dip and culvert construction, performance, and maintenance could be assessed.

Sites were sought that would provide variety in some of the factors, but be similar in others so that the effects of individual variables on drainage device performance could be isolated.

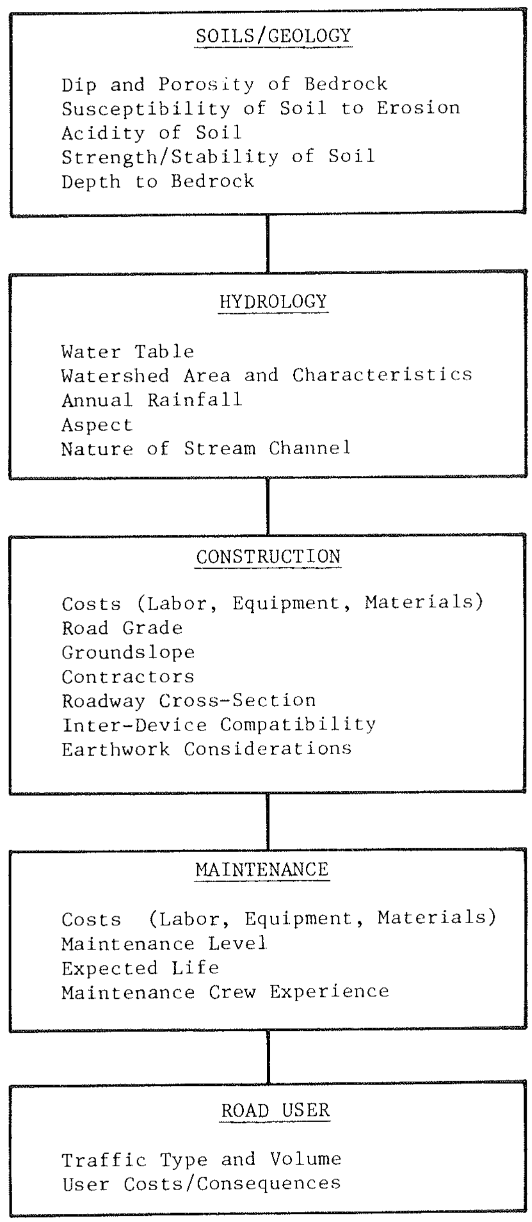

FIGURE 2 Preliminary framework of factors to consider in the selection of broad-based dips or culverts.

Another criterion was that plans and drawings had to be available for the roads selected for study. It was also thought to be desirable that the study roads be associated with active timber sales, although this was not mandatory.

Given these criteria, the investigators met with engineers, soil scientists, and timber sales personnel from the Monongahela National Forest Headquarters in Elkins, West Virginia, to discuss specific study sites. Although a large number of sites were proposed by U.S. Forest Service (USFS) personnel, time and resource constraints limited the actual number to 19. A summary listing of the characteristics of each site is provided in Table 1. Plans, drawings, and appropriate U.S. Geological Survey (USGS) quadrangle sheets were acquired for each of the study sites. Average annual precipitation was estimated from published information (*6*). These items were used to assist in planning the field work and to supplement the data acquired in the field.

Once the field sites were identified, efforts centered on the selection of specific data items to be collected in the field. Information was sought in regard to each study site as a whole and to the individual drainage structures that comprised a site. Data pertinent to the overall site included average annual rainfall, general soil conditions, length, number of broad-based dips, number of 18-in aluminum pipe culverts, traffic volume data, timber sales data, and actual project construction and maintenance costs. Both average daily traffic (ADT) and total number of truck trips necessary for timber removal were sought in the traffic category. The number of truck trips could be computed given the volume of timber sales and the typical number of board feet of timber per truck load.

Information pertinent to individual drainage structures included spacing from a previous structure, type of upgrade structure, road gradient, roadway cross-section, ground slopes, aspect, watershed location, fill height, presence of cut-fill transitions, cross-drain purpose, road surface type and depth, horizontal curvature, and structure performance. Actual device construction and maintenance costs were sought whenever possible.

A study procedure was developed to acquire the desired data efficiently. This procedure basically consisted of a two-person field party that walked over the roads of the study site, recorded the previously mentioned qualitative roadway and environmental information, and made simple measurements to quantify certain roadway and drainage parameters. About 3 weeks were required to complete the structural performance field study.

Two types of data collection forms were prepared and tested to document the information collected in the field. The first form consisted of a summary of information that related to the

TABLE 1 CHARACTERISTICS OF ROADS IN MONONGAHELA NATIONAL FOREST THAT WERE SELECTED FOR FIELD STUDY

Project Sites (USFS Road Name and Number)	Length (miles)	Number of Broad-Based Dips	Number of 18-inch Aluminum Culverts	Avg. Annual Rainfall (Inches)
1. Stuart Camp, 91-B	0.09	2	1	49.0
2. Stuart Camp, 91-C	0.43	9	0	49.0
3. Suart Camp, 91-D	0.32	6	1	49.0
4. Stuart Camp, 319	0.10	2	0	49.0
5. Peach Orchard, 297	0.77	6	1	38.7
6. Four Mile, 969	1.31	14	11	45.7
7. Sue, 796	0.63	10	0	45.7
8. Sue, 796-C	0.95	11	7	45.7
9. Music Run, 907	1.22	0	20	59.7
10. Hacking Run, 914	0.52	6	7	56.5
11. Galford Run, 90	3.16	0	41(15)[a]	47.1
12. Galford Run, 90-A	3.30	39(15)	16(5)	47.1
13. Stony Run, 757	1.54	17	13	47.1
14. Divide, 790	1.77	0	8	47.1
15. Leatherwood, 368	6.52	34(7)	54(13)	54.6
16. Jobs Run, 117	2.35	46	13	53.8
17. Lick Drain, 929	4.40	47	29	53.8
18. Red Run, 244	3.60	0	67	53.8
19. Warner Run, 916	3.64	63	16	50.1

[a]Numbers in parenthesis represent number of drainage structures studied for those instances where it was not possible to survey all structures.

overall project. One form was completed for each project. The form was designed to be completed by Forest Service personnel and the researchers (using the maps, plans, and drawings furnished by the Forest Service) before the field surveys were conducted.

The second data form was completed in the field by the two-person study team. Information about the location and environment of each individual drainage structure was recorded. One form was used for each structure. It should be noted that because of time, resource, and weather constraints, a complete set of data could not be acquired for all sites.

Performance was rated as either acceptable or unacceptable by the survey team after completing an examination of the drainage structure. Unacceptable structures were those that required, in the opinions of the field party, immediate maintenance attention. When a structure was determined to be unacceptable, a note was made of its distress type, which could have been one of the following:

- Rutting of dip or roadway,
- Siltation of dip,
- Erosion of fill slope,
- Corrosion of pipe,
- Ponding of dip,
- Clogging of inlet or pipe,
- Sloughing of cut-slope, or
- Construction problems.

Before the field work was begun, special efforts were made to ensure that the survey team understood the types of distress. Distress types were defined both in words and by photographs (taken during the field trips) that depicted an example of a particular condition.

The format of the data on soils, which consisted of such items as soil name, geologic formation, soil erosion potential, and other information, was developed based on unpublished USFS guidelines of soil characteristics for drainage and road building in the Monongahela National Forest. The soil found at each cross-drain was identified by use of a combination of descriptions of soil color and texture made in the field, and information from USDA county soil surveys and USFS documents. Once the soil types and geologic formations were identified, the USFS rating guide for soil sensitivity groups in Monongahela National Forest was used to determine relevant soil characteristics. However, the coarse fragment content of the soil was determined visually. Estimates of depth to bedrock and depth to seasonal high water table had to be obtained from USDA county soil surveys.

DATA ANALYSIS

The data were initially categorized in such a manner that the performance of 18-in metal culverts could be examined separately from the performance of broad-based dips. Of the 482 structures studied, 227 were culverts and 255 were broad-based dips. When study sites were selected, an attempt was made to examine an approximately equal number of dips and culverts. Seventeen of the 227 culverts were rated as unacceptable, which represents a failure rate of 7.5 percent. The overall failure rate for dips was 27.5 percent. The performance of the culverts that were studied was therefore substantially better than that of the dips.

The distress types noted for the culverts that were rated as unacceptable were sloughing of the cut slope, clogging of pipe or inlet, and erosion of the fill slope. The most serious problems were sloughing of the cut slope and clogging of the pipe or inlet. Sixteen of the 17 failures involved sloughing problems. Fifteen of the failed culverts were clogged; only three culverts were noted to have erosion of the fill slope. Sloughing of the cut slope and clogging are related distresses; the material that sloughs off the slope gets deposited in the inlet or pipe and clogs the structure.

The most common distress types for the broad-based dips that were rated as unacceptable were erosion of the fill slope, rutting, siltation, and ponding. Of the 70 failed dips, 48 were noted for erosion of the fill slope, 35 for rutting problems, 34 for siltation, and 27 for ponding. Four dips were noted for construction-related distress. Only one dip that failed was noted for sloughing of the cut-slope.

In order to assist in specifically determining which factors affected drainage structure performance, the data were organized into the following groups: design factors, soil and geologic factors, hydrologic factors, and traffic factors. The various factors that comprised these categories were analyzed individually to determine if each had a substantial effect on structure performance.

Substantial factors were those that were judged by the researchers to yield a relatively high failure rate; all other factors were considered secondary. This terminology was arbitrarily selected by the investigators for their convenience in describing the data tabulations; no statistical significance should be attached to the results.

The researchers used a statistical method known as the normal approximation to the binomial to compare the overall performance of culverts and dips with the performance of these devices when categorized by the various factors (*7*). Plots were prepared by use of the appropriate statistical equation and the aforementioned overall structure performance rates for 18-in culverts and broad-based dips. The plots depicted statistical significance as a function of the number of failed and number of total structures within a given sample set. The following three statistical regions were identified:

- Values that indicated a significantly better structural performance when compared to the overall situation,
- Values that indicated a significantly worse structural performance, and
- Values that indicated no significant change in structural performance.

Design Factors

The following items were considered as design factors:

- Road grade,
- Roadway cross-section,
- Structure spacing,
- Road surface type,
- Immediate up-grade (on-roadway) structure,
- Fill height,
- Horizontal curvature,
- Cross-drain skew, and
- Cut-fill transitions.

Appropriate groups of data were developed for each factor and the frequency of device failure was determined for the groups. Roadway cross-section and cross-drain skew were determined not to have a substantial effect on drainage device performance.

An examination of structure performance versus road grade indicated that culverts performed best when the road grade was 7 percent or less. The culvert failure rate was only 2.5 percent for road grades less than or equal to 7 percent. The failure rate was 13.1 percent for road grades greater than 7 percent. Broad-based dips demonstrated a somewhat comparable trend in that they also performed better as the road grade decreased. However, dips tended to perform best when the road grade was less than or equal to 3 percent. The dip failure rate was 32.3 percent for grades between 3 and 9 percent.

Drainage structure spacing is a design factor that depends on the road grade. Mean spacings for acceptable and unacceptable culverts and dips, respectively, are presented in Figures 3 and 4 for different road grades. Those devices that did not have an adjacent drainage structure were deleted from this particular analysis. The spacing of failed structures was generally greater than the spacing of acceptable structures. The data also indicated that the mean spacing for acceptable culverts was relatively close to the design spacing value. The mean spacings for unacceptable dips deviated more widely from the design value.

Structure performance versus road surface type was also examined. As was expected, broad-based dips generally performed better when armored with gravel than they did in an unsurfaced condition. The failure rate for unsurfaced dips was 37.3 percent compared to 19 percent for dips armored with stone. The data were insufficient to evaluate which type of gravel surfacing performed better because only seven dips were armored with 3-in quarry stone. Based on information acquired during practitioner input, there appeared to be some disagreement between engineers as to whether 3/4-in crusher run or 3-in quarry stone was a more appropriate surfacing for logging roads in the Appalachian region. Additional research into this issue could prove to be fruitful.

Culverts or dips located on horizontal curves had a higher frequency of failure than structures located on tangent sections of roadway. However, dip performance appeared to be more adversely affected by curvature than that of culverts. The failure rate for dips located on curves was 40 percent, compared to about an 8 percent failure rate for culverts located on curves.

Fill height is another design factor that affects structure performance. The failure rate generally increased as the height of the fill increased for both dips and culverts. Closer examination indicated that the failure rate increased dramatically for dips when the fill height was greater than 3 ft, as shown in Figure 5. The failure rate for fill heights less than or equal to 3 ft was 18.9 percent, compared to a rate of 67.2 percent for fill heights greater than 3 ft.

Culvert and dip performance versus type of drainage structure immediately up-grade from the one in question was investigated. Drainage structures that were located in a sag were separated from the other structures in this analysis. This was done in order to specifically examine structure performance at sag locations, which practitioners had indicated were locations that were critical to dip performance. The analysis indicated that sag locations had a 45.5 percent failure rate for broad-based dips and a 9.7 percent rate for culverts. Based on these results, it is recommended that if it is necessary to locate a drainage structure in a sag location, a culvert should be used because it will probably perform better than a broad-based dip.

Although it was difficult to determine from the data which type of structure was better in terms of drainage, it was noted that dips were more sensitive than culverts to the existence of an up-grade drainage structure. When no structure was located up-grade from a dip, the failure rate was 41.1 percent. By contrast, when no structure was located up-grade from a culvert, the failure rate was only 8.7 percent.

Notably few study site drainage devices were located at cut-

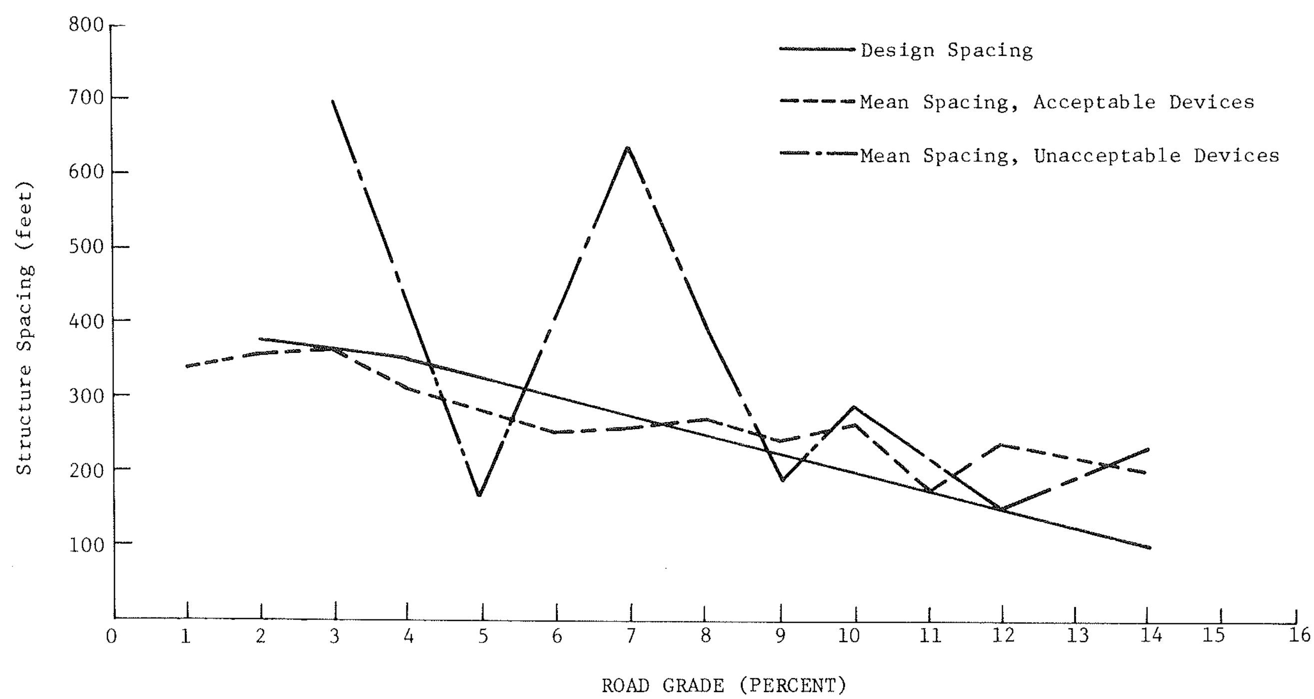

FIGURE 3 Mean culvert spacing versus road grade.

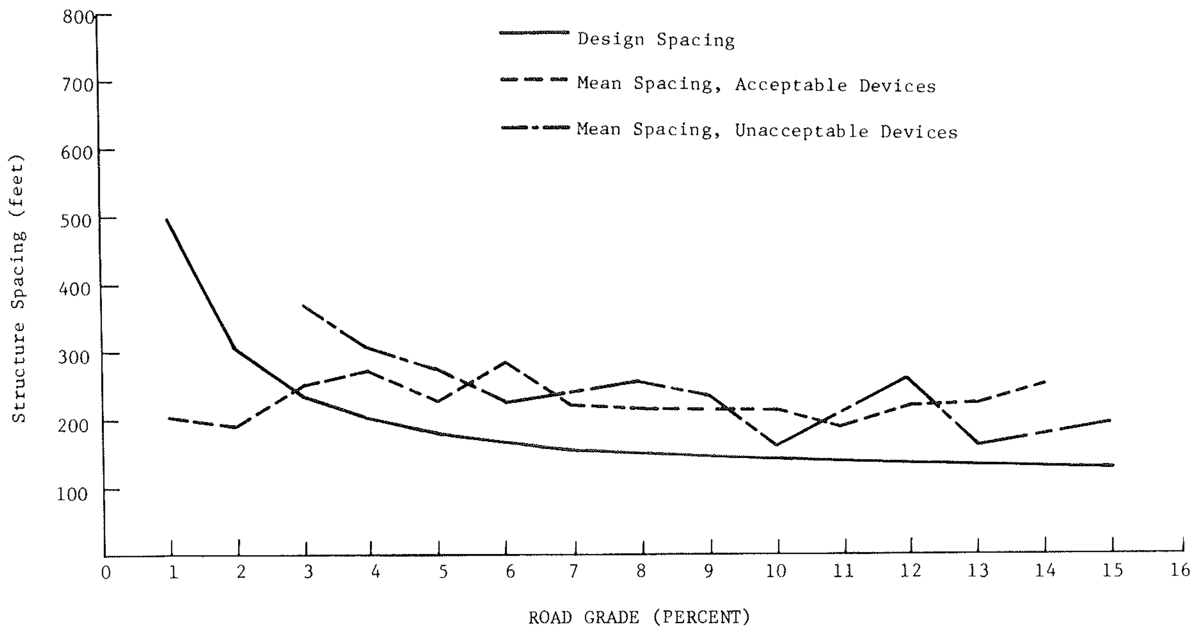

FIGURE 4 Mean spacing of broad-based dips versus road grade.

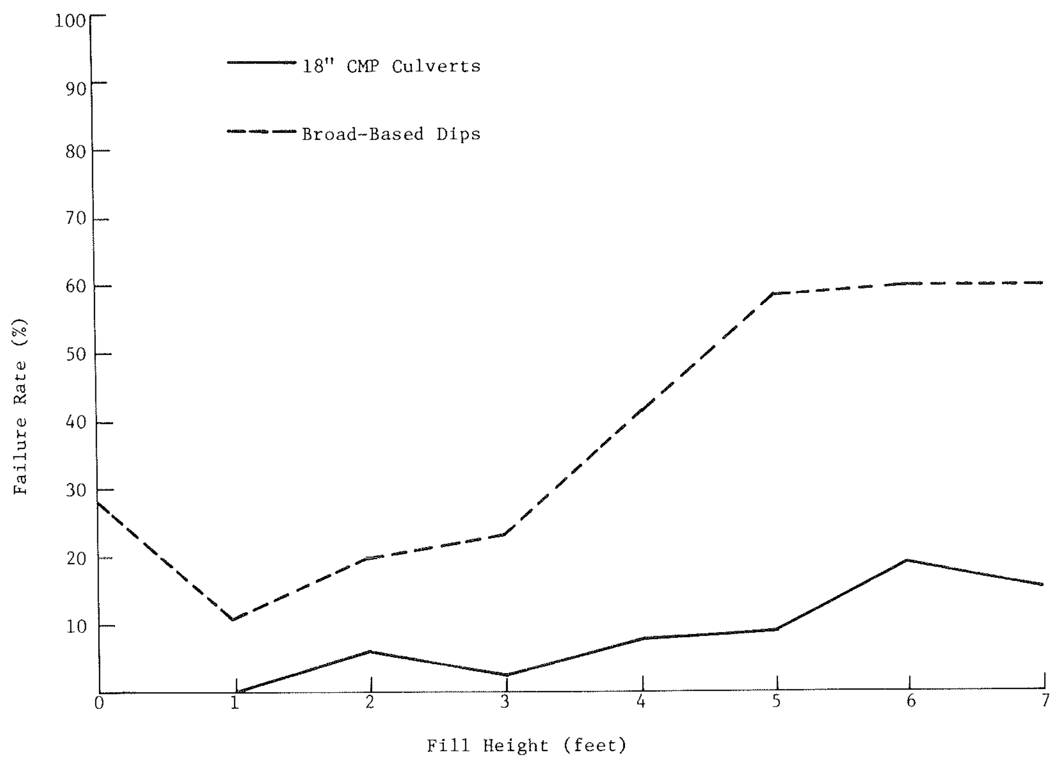

FIGURE 5 Drainage structure performance versus fill height.

fill transitions. This may be an indication that designers have learned from experience to avoid placing drainage structures at cut-fill transitions. Based on a very small sample size, it appears that culverts should be used when drainage structures must be located at cut-fill transitions. Although only four dips were located at cut-fill transitions, two of them failed. However, none of the five culverts at cut-fill transitions failed.

It should be noted that a limitation of the analysis just described was the assumption that all drainage grades, frequency and amount of flow, and other parameters of this nature were the same for the sites under consideration. A comparison of failure rates based on a single variable, such as whether or not an up-grade drainage structure exists, could be misleading unless other flow characteristics are considered.

Soil and Geologic Factors

The following items were considered soil or geologic factors:

- Soil type,
- Geologic formation,
- Soil erosion potential,
- Ground slope,
- Suitability of soil as road material,
- Coarse fragment content, and
- Depth to bedrock.

Soil erosion potential, soil suitability for road building, coarse fragment content, and depth to bedrock were analyzed but were determined not to be significant factors in drainage structure performance.

Structure performance versus specific type of soil was examined initially. However, the number of different soil types studied was so large that it was difficult to determine which soils contributed to acceptable or unacceptable structure performance. Therefore, soils were reclassified according to soil series. For example, any soil type that had Berks in its title was classified as a Berks soil. The performance of 18-in metal culverts was essentially independent of soil series. However, certain soils tended to contribute to poor dip performance. Soils associated with high failure rates for broad-based dips were generally gravel-sand-silt mixtures, gravel-sand-clay mixtures, sand-silt mixtures, and silty or clayey fine sands that were typically derived from interbedded sandstone, siltstone, and shale.

It should be noted that the soil data collected in this study were admittedly general in nature. Laboratory testing would have been desirable to determine specific reasons why the specific soils series had such high failure rates. It is believed that the high failure rates may be related to the fineness of these soils, the poor bearing strength of shale fragments present in several of these soils, and other characteristics of the underlying geologic formation.

Because soil characteristics are closely related to and vary with the underlying geologic formation, an analysis was made of the influence of geologic formation on drainage structure performance. This analysis revealed that culvert performance is independent of geologic formation. Dips, however, demonstrated high failure rates in the interbedded sandstone, siltstone, and shale formations.

The ground slope was also examined because of its suspected relevance to structure performance. Because water is transmitted to drainage structures from higher elevations, it was believed that the ground slope above the drainage structure was critical. Culvert performance worsened as the ground slope increased. However, the same trend was not true for dips.

The poor performance of culverts on steeper ground slopes may result from material such as rocks, limbs, and other unstable materials rolling down these steep slopes and becoming lodged in the culverts. High cut slopes, which are undesirable for culverts, may be needed when the ground slopes are steep. Because dips are not enclosed like culverts, they do not exhibit these characteristics, and are not as dependent on changes in the ground slope as culverts.

Hydrologic Factors

Hydrologic factors affect the quantity of water that passes through a drainage structure. These factors are obviously an important component of dip and culvert performance. The following items were considered hydrologic factors:

- Aspect,
- Watershed location,
- Average annual precipitation,
- Presence of seeps or springs,
- Cove location,
- Soil wetness,
- Surface water yield,
- Ground water yield, and
- Depth to seasonal high water table.

Soil wetness, surface water yield, ground water yield, and depth to seasonal high water table were examined, but no substantial differences between devices were indicated.

A limitation of the data was that the aspect was classified as either north or south to ease the tabulation of data, and because these exposures had been identified by practitioners to be an important determinant of dip performance. However, the classification of aspects into such broad categories could have influenced the results of the study. For example, a drainage structure located on an east-northeast exposure could have received the benefit of the drying effect of solar radiation because its exposure was closer to the east than to the north. However, in this study the exposure would have been classified as a north aspect.

Results indicate that culverts and dips located on a north aspect both had higher failure rates than those with a southern exposure. This can be attributed to moisture- and water-related problems that are exacerbated by the lack of solar radiation and its drying effect. The failure rate for dips with a north aspect was not as great as had been anticipated, which could be a result of the data limitation just described.

Watershed location was another factor that was studied. The location of the drainage device in the watershed was classified as belonging to either the upper, middle, or lower third of the watershed. The purpose of this breakdown was to provide a rough estimate of the relative amount of runoff handled by the device. As the watershed location went from high to low, failure rates increased for both culverts and dips. This suggests that the greater the volume of water handled by the drainage structure, the greater the likelihood of failure.

The average annual precipitation indicates the variability in the quantity of water handled by drainage devices from project to project. The analysis of this factor was made difficult by the fact that rainfall data were estimated using available meteorological information instead of collecting specific data from each site. Two study roads would therefore be assigned the same quantity of precipitation that was determined from the nearby weather station although they were several miles apart and probably received slightly different amounts of rainfall. However, it was believed that the relative, rather than the absolute, amount of annual precipitation would be of greater value in this study.

Culverts and dips both experienced an increase in failure rate as the average annual precipitation increased. This result had been expected, because structures in regions of high precipitation carry large volumes of runoff, and are prey to ground water or moisture-related problems.

Two locations that tend to cause problems for drainage structures were identified from the literature review and practitioner input. Drainage structures located in coves or where springs or seeps are present will be generally exposed to larger quantities of water than they would at other locations. The Forest Service tends to avoid locating dips where springs, seeps, or coves exist. It was the opinion of the researchers that the reason three of the twenty-two 18-in aluminum culverts failed where seeps or springs were present was that these pipes were undersized. A larger pipe would probably have functioned properly in these three locations.

Traffic Factors

One would intuitively expect drainage structure performance to be related to the magnitude of traffic using the roadway. Data were available in this study for average daily traffic and total truck traffic. Values for ADT were determined from estimates furnished by forest rangers for the projects within their particular jurisdiction of the Monongahela National Forest. The data indicated that the volume of traffic did not affect culvert or dip performance. This latter finding was unexpected; however, the available data did not yield a specific explanation. Because broad-based dips are actually part of the road surface, it had been hypothesized that an increase in traffic would correlate with an increase in the failure rate for dips.

Perhaps a better measure of the traffic load applied to a drainage structure is the total number of truck trips made to date on the logging road. The total number of truck trips was derived by dividing the quantity of timber (in board-feet) involved in the timber sale by the average number of board-feet of lumber per truckload. Therefore, in this case traffic volume could be considered a surrogate measure for the weight applied to the roadway. Both the number of vehicles and their weights have an impact on roadway and drainage structure performance.

Culvert and dip performance were not affected by changes in traffic volume. This finding was unexpected. An important factor for which data were not available in this study was the condition of a dip when it received traffic. A dry dip can withstand many more load applications than a wet one. It is hypothesized that the amount of truck traffic could be an important factor in the prediction of dip performance under certain conditions. Additional research is warranted in this area.

THE DIP VERSUS CULVERT DECISION

Information was provided in the preceding section on the performance of metal culverts and broad-based dips in relation to certain design, soil, hydrologic, and traffic factors. An earlier study compared culverts and dips on an economic basis in terms of construction, maintenance, and road user costs (*5*). The findings of each of these aspects in the overall study were combined to develop guidelines that would assist in the identification of those conditions in which aluminum pipe culverts are more appropriate than broad-based dips for intermittent cross-flows on Appalachian logging roads. The results are presented in this section in the form of a guided decision-making scheme. It should be noted that the guidelines (and the research findings from which they were developed) are based on dips and culverts that were constructed to USFS standards. The guidelines are not directly applicable to drainage devices that were not constructed to these standards.

The approach taken was to identify those forest road situations in which broad-based dips were not an appropriate drainage device. By a process of elimination, a drainage location that did not possess any of these characteristics would be a likely candidate for the installation of a broad-based dip. A list was developed of conditions that had been identified in the drainage device performance study as being strongly associated with poor dip performance. The traffic volume criteria from the economic analysis and certain other conditions that had been identified from practitioner input were added to this initial list. This list of conditions formed the decision-making framework for the dip versus culvert decision. A review of the list indicated that a flowchart might be the most appropriate format for presenting the guidelines on dip and culvert usage. The decision-making flowchart is shown in Figure 6.

The user begins the decision-making process by identifying the drainage location of interest on a logging road and noting its characteristics. Characteristics for which information is needed include:

- Road surface type,
- Volume and type of vehicular traffic,
- Hauling schedule,
- Stream characteristics,
- Water table characteristics, and
- Roadway design elements, such as curvature, grade, and cut-fill characteristics.

By considering one drainage location at a time and answering "yes" or "no" to questions about the characteristics just noted, the analyst follows the flowchart until one of two possible outcomes is reached: (a) a culvert is recommended for the particular location or (b) use of a broad-based dip appears feasible. In the latter case, a soil scientist or geotechnical engineer should be consulted for advice on the suitability of the soil for dips. Although it was initially intended to develop specific soil and geologic criteria for dip installation, this proved to be impossible for several reasons. Although a variety of soil and geologic combinations had been examined in the field study, insufficient data existed about any single combination to permit firm conclusions to be drawn. The results were similarly limited because they were based on a relatively cursory field observation of soil characteristics instead of on a more thorough or quantitative analysis of soil properties. Finally, the type of soil data the typical user would have available was not known.

The traffic volume criteria that appear in the guidelines were based on results of the economic analysis (*5*). It is recommended that culverts should strictly be used on any road that carries an excess of 15 vpd no matter if it is subjected to this traffic level for 2 or 20 years, because of the high user cost associated with dips. Dips are appropriate on roads with traffic volumes less than 5 vpd, assuming that their use is not precluded by a design, soil, or hydrologic factor. For traffic volumes between 5 and 15 vpd, the decision to use a dip or culvert should be influenced by how much the road is used by log-hauling vehicles. If the road is used each year for the life of the road (assumed to be 20 yrs), the high road user costs associated with broad-based dips make culverts the preferred drainage device. In cases in which the road is to be used for timber harvesting only during the first few years of its life and then closed for a period of time, broad-based dips are the more economical drainage structure.

Once the decision of whether to use a dip or a culvert has been made, the user should repeat the process by identifying the next

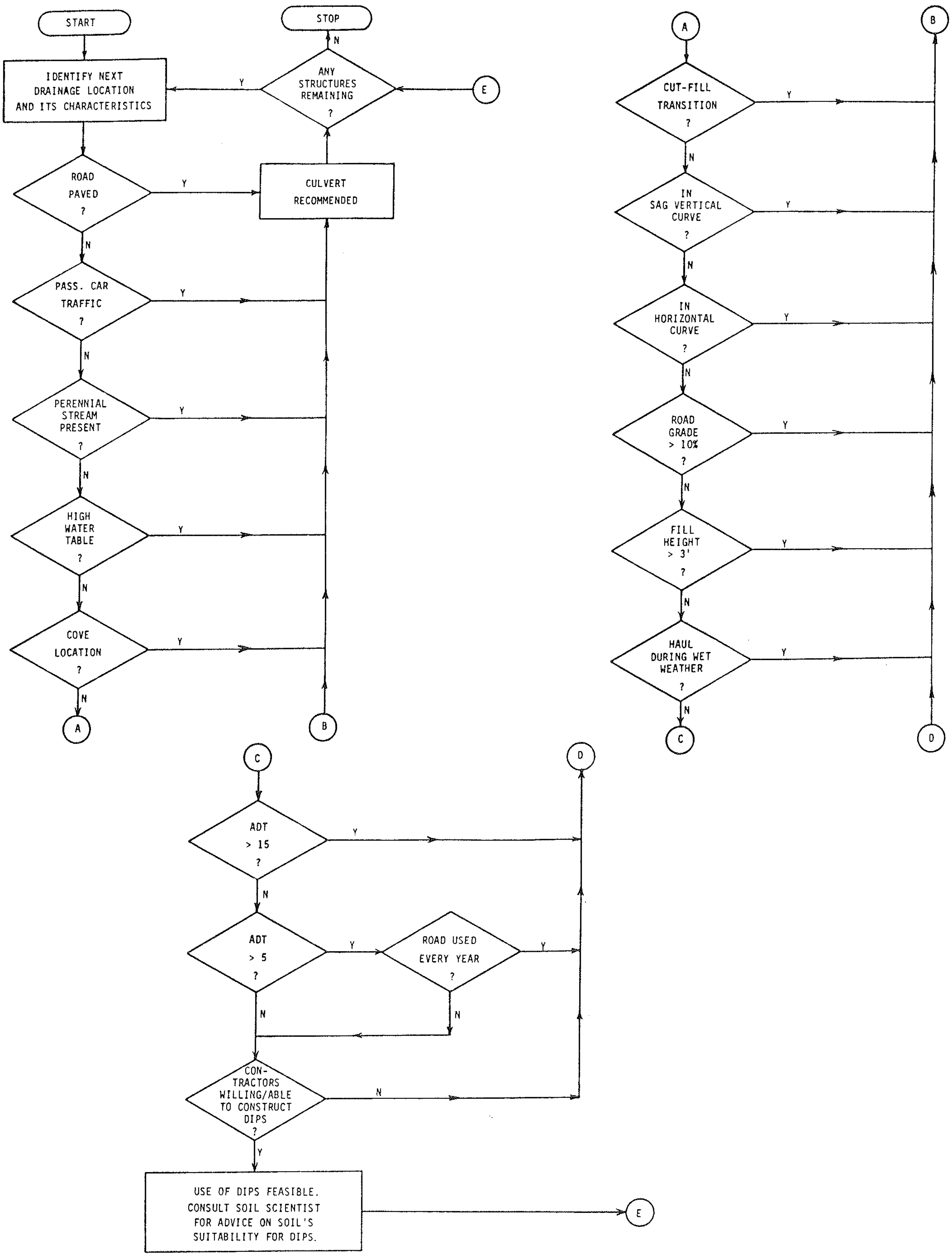

FIGURE 6 Decision-making flowchart for the selection of either broad-based dip or 18-in aluminum culvert for use on forest roads in the central Appalachian region.

drainage location on the road and proceeding through the list of questions again. This procedure should continue until all drainage locations are evaluated. It should be noted that the decision-making framework presented here is based on the assumption that the optimal design of the road will include a mix of dips and culverts. There may be certain situations in which a road may be built in which dips or culverts are used exclusively for legitimate reasons. For example, if local contractors have neither the ability nor the desire to build broad-based dips, all drainage devices on the road would be culverts.

It must be emphasized that the flowchart described earlier is only a guide or aid in the selection of the type of drainage to use on forest roads. The user's experience and familiarity with haul road design and drainage, and with the region in which the road is located, will play a major role in determining how effectively the flowchart meets its intended objectives. It must also be kept in mind that the development of the flowchart was mostly based on data from the central Appalachian region. Users in other regions of the country may find certain items on the flowchart inappropriate or may feel a need to include an additional decision-making capability. Such modifications can be handled relatively easily and would increase the flexibility of the flowchart as a decision-making tool. For example, the flowchart presented here applies to 18-in, corrugated aluminum pipe culverts. If steel culverts are being considered, it might be appropriate to check the pH of the soil. An acidic soil could significantly shorten the life of a steel culvert.

CONCLUSIONS AND RECOMMENDATIONS

Neither dips nor culverts are a panacea for drainage problems on logging roads; each device has its unique strengths and limitations. However, certain situations exist in which one device may be more appropriate and cost-effective than the other. These conditions have been documented, and a decision-making framework was developed to assist engineers or foresters in selecting the appropriate drainage device for a particular application.

The performance of the culverts in this study was substantially better than that of broad-based dips. Many of the dip failures could be traced to one or more underlying factors that, in retrospect, made the installation of a culvert the more appropriate solution for that location. Culverts, however, generally failed as a result of the pipe or inlet clogging. This demonstrates the importance of a regular culvert inspection and maintenance program to identify and correct problems before they reach destructive levels. Other, more specific conclusions that were drawn from the drainage structure field performance study are as follows:

- Drainage device spacing guidelines that were found in the literature are appropriate for the central Appalachian region. Wider spacing of dips and culverts to reduce costs is not recommended because failure rates increase dramatically.
- Dips armored with gravel perform better than unsurfaced dips.
- Dips should not be installed at horizontal curves.
- Dips should not be installed on fills more than 3 ft high.
- Culverts are preferable to broad-based dips in sag locations.
- The performance of 18-in, corrugated metal pipe culverts is relatively independent of soil characteristics. Dip performance, however, is closely related to the soil's erodibility and other characteristics. The advice of a soil scientist should be sought before a broad-based dip is recommended for a particular location.
- Dip performance is independent of ground slope. Culverts, however, are prone to clogging when ground slopes exceed 45 percent.
- Culverts located on road grades of less than or equal to 7 percent should perform effectively if they are constructed and maintained in accordance with recommended USFS guidelines.

The results of this study unexpectedly indicated that culvert and dip performance was not affected by traffic volume. An important factor for which data were unavailable was the condition of a dip when it received traffic. A dry dip can withstand many more load applications than a wet dip. It would be desirable to extend the results of this study by examining the relationship between traffic volume, dip condition (wet or dry), and structural performance.

Although the results of this study were based on very general soil data, they demonstrated the important role that soil and geologic factors play in the prediction of drainage structure performance. An additional, in-depth study by a soil scientist or geotechnical engineer of specific soil and rock types and their relation to drainage structure performance is warranted.

Broad-based dips armored with gravel generally perform better than unsurfaced dips. The data were insufficient to evaluate which type of gravel surfacing performed better because only a few dips were armored with 3-in quarry stone; the rest were surfaced with crusher-run stone. Based on information acquired during practitioner input, there appeared to be some disagreement between road designers as to whether 3/4-in crusher-run stone or 3-in quarry stone was a more appropriate surfacing for logging roads in the Appalachian region. Given the current high cost of road surfacing materials, additional research into this issue could be fruitful.

ACKNOWLEDGMENTS

This paper is based on research supported by funds provided by the U.S. Department of Agriculture, Forest Service, Northeastern Forest Experiment Station. Special thanks are extended to Randall Burks, who played a major role in the field data collection. Appreciation is also expressed to the staff of the Monongahela National Forest for their assistance and cooperation throughout the study.

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