CONFERENCE PROCEEDINGS 6

Sixth International Conference on
Low-Volume Roads

Volume 2

TRANSPORTATION
RESEARCH
BOARD

NATIONAL
RESEARCH
COUNCIL

CONFERENCE PROCEEDINGS 6

Sixth International Conference on Low-Volume Roads

Volume 2

Minneapolis, Minnesota
June 25–29, 1995

Sponsored by
Forest Service, U.S. Department of Agriculture
Federal Highway Administration, U.S. Department of Transportation
Bureau of Indian Affairs, U.S. Department of the Interior
Agricultural Marketing Service, U.S. Department of Agriculture
Kuwait Fund for Arab Economic Development

TRANSPORTATION
RESEARCH
BOARD

NATIONAL
RESEARCH
COUNCIL

NATIONAL ACADEMY PRESS
WASHINGTON, D.C. 1995

Conference Proceedings 6
ISSN 1073-1652
ISBN 0-309-06074-5

Subscriber Categories
I planning, administration, and environment
II design
III materials, construction, and maintenance
IV operations and safety

Transportation Research Board publications are available by ordering individual publications directly from the TRB Business Office or by annual subscription through organizational or individual affiliation with TRB. Affiliates and library subscribers are eligible for substantial discounts. For further information or to obtain a catalog of TRB publications in print, write to Transportation Research Board, Business Office, National Research Council, 2101 Constitution Avenue, N.W., Washington, D.C. 20418 (telephone 202-334-3214).

Printed in the United States of America

The conference was sponsored by the Forest Service, U.S. Department of Agriculture; Federal Highway Administration, U.S. Department of Transportation; Bureau of Indian Affairs, U.S. Department of the Interior; Agricultural Marketing Service, U.S. Department of Agriculture; and the Kuwait Fund for Arab Economic Development.

Contents

TECHNOLOGY TRANSFER

VOLUME 2

MATERIALS

WASTE MATERIALS

Foreword

This conference marks the 20th anniversary of the Transportation Research Board (TRB) specialty conferences on low-volume roads. Two decades of concentrated efforts have helped solve problems and provided opportunities for low-volume road planning, design, construction, maintenance, operations, management, and evaluation. Also addressed over the past 20 years have been other low-volume road system concerns, such as economics, the environment, pavements, other surface treatments, stabilization, materials, bridges, and safety. These conferences have also provided, along with their proceedings, a tremendous technology transfer effort.

The technology transfer effort fulfills the consensus of the First International Conference [Workshop] on Low-Volume Roads, held June 16–19, 1975, in Boise, Idaho. The late Eldon J. Yoder, in whose honor outstanding paper awards are named, provided the leadership for that conference. In the introduction to the proceedings (*Special Report 160*, pp. 1–3), he stated, "A continuing activity should be undertaken by the Transportation Research Board with the objectives of developing information, providing an information exchange forum, and disseminating the findings in such a manner as to be most productive toward an early solution to this pressing problem." The problem to which he referred was "the need to revitalize this long-neglected system . . . [of low-volume roads]."

Since 1975, conferences on low-volume roads have been held every 4 years: 1979 in Ames, Iowa; 1983 in Phoenix, Arizona; 1987 in Ithaca, New York; 1991 in Raleigh, North Carolina; and 1995 in Minneapolis, Minnesota. The science of low-volume roads has come a long way since that first conference when research papers ranged from "Dilemmas in the Administration, Planning, Design, Construction, and Maintenance of Low-Volume Roads" (*Special Report 160*, pp. 7–16) to "Methodology for Establishing the Economic Viability of Low-Volume Roads" (*Special Report 160*, pp. 385–395). In contrast, the papers and technical notes for the sixth conference published in these proceedings address up-to-date topics on who is doing research and technology transfer on low-volume roads, what is being done to improve low-volume roads, where low-volume road improvements are taking place, when additional research results are expected, and how information on low-volume roads is being disseminated.

The two volumes of these proceedings constitute the combined efforts of hundreds of people, including the authors, to whom goes the greatest recognition and to whom we are most indebted, and the Conference Steering Committee, which spent several days reviewing paper and technical note synopses, selecting those that appear in these proceedings, and organizing the draft format of the conference program. In addition, more than 100 people assisted in the paper and technical note peer review process, including members of the Conference Steering Committee, members and friends of the TRB Committee on Low-Volume Roads, staff of the Institute for Transportation Research and Education of North Carolina State University, and TRB staff, particularly G. P. "Jay" Jayaprakash and Catherine Spage.

As did previous conferences, this Sixth International Conference on Low-Volume Roads provides a unique opportunity for engineers, planners, administrators, practitioners, and researchers to exchange information and benefit from recent research related to low-volume roads. However, this conference would not have been possible without the financial support of the conference sponsors: Forest and Agricultural Marketing Services, U.S. Department of Agriculture; Federal Highway Administration, U.S. Department of Transportation; Bureau of Indian Affairs, U.S. Department of the Interior; and the Kuwait Fund for Arab Economic Development.

These sponsors provided funding to TRB for the editing and printing of these proceedings and to help organize, publicize, and manage the conference. We thank all of them for their support.

Of course, the conference cohosts—the Center for Transportation Studies at the University of Minnesota, the Finnish National Road Administration, the Transportation Association of Canada, and the Minnesota Department of Transportation—deserve much recognition and our thanks for providing excellent facilities and staff support for the conference, not only for the technical sessions and the field trip but also for registration, travel, accommodations, additional technical tours, software and videotape demonstrations, the family and guest program, other social events, and information on the Twin Cities area and the state of Minnesota.

I feel personally indebted to all of the people and agencies who helped make this landmark conference a great success. I thank each and every one of you.

Robert L. Martin
Chairman, Steering Committee for the Sixth International Conference on Low-Volume Roads

MATERIALS

Prevention of Salt Damage to Thin Bituminous Surfacings: Design Guidelines

B. Obika, *Botswana Roads Department*
R. J. Freer-Hewish, *University of Birmingham, United Kingdom*
M. Woodbridge and D. Newill, *Transport Research Laboratory, United Kingdom*

A design method to prevent soluble salt damage to thin bituminous road and runway surfacings based on laboratory tests and field trials is proposed. Remedial treatments for damaged surfacings are also suggested. The damage, characteristic of warm islands and arid and semiarid inland regions of the world, occurs when salts crystallize in the pavement, physically disrupting the bituminous surfacing and causing premature deterioration of the road. Bituminous primes have been found to be more susceptible to damage than final surface dressings. The design method has been developed from laboratory studies in the United Kingdom and field trials in the West Indies and Botswana. They apply wherever a waterbound or a chemically stabilized pavement layer is covered with a thin bituminous surfacing.

Soluble salts in pavement layers, groundwater , or both can migrate upward to the surface and damage the thin bituminous surfacing. This migration through capillary action is mainly caused by evaporation at the surface. At or near the surface, the salts in solution become supersaturated and crystallize. This creates pressures with associated volume changes that can lift and physically degrade the bituminous surfacing and break the adhesion with the underlying pavement layer. The damage may appear in the form of "blistering," "doming," "heaving," "fluffing," and "powdering" of the bituminous layer.

Soluble salt is defined (*1*) as "basically those minerals that are most soluble, notably salts of magnesium and sodium." Gypsum (calcium sulfate), an abundant salt, is only slightly soluble and does not cause damage. This type of damage has been reported in southern and western Australia, southern and northern Africa, West Indies, India, Chile, United States, and the Middle East. By superimposing the location of recorded cases of surfacing damage from soluble salts onto Meig's (*2*) classification map of world climates, it is apparent that salt damage of bituminous surfacings is restricted mainly to warm islands and arid and semiarid inland regions where evaporation exceeds precipitation. Moisture is drawn to the surface where any soluble salts present can be precipitated.

The published work on soluble salt damage to bituminous pavements was reviewed by Obika et al. (*3*). The published papers deal with local climates and materials, and the recommendations cannot necessarily be universally applied. Whereas very small amounts of soluble salt (0.2 percent) can cause damage, higher percentages have been used without subsequent damage.

There appears to be a requirement to standardize methods of salt content analysis so that maximum salt limits can be compared. The sulfate values of Weinert and Clauss (*4*) are based on the analysis of water extracts, whereas Fookes and French (*5*) refer to acid-soluble sulfates. Netterberg et al. (*6*) quote sulfates as SO_4 and others [e.g., Weinert and Clauss (*4*) and Netterberg (*7*)] as SO_3. To relate SO_3 to SO_4, multiply the SO_3 value by 1.2. Cole and Lewis (*8*) used NaCl, and Weinert and Clauss (*4*) measured chloride.

The recommended salt limits refer to the initial salt content of the materials used in the pavement structure. However, tests have shown that, depending on temperature and relative humidity, the final salt content at the pavement surface before surfacing can be significantly higher than the initial amount. Ideally, it is desirable to specify the maximum salt content at the surface before bituminous surfacings. Practically, this value may have to be estimated as discussed later.

The design criteria developed in this paper are based on a review of previous work, laboratory simulations in environmental cabinets at the University of Birmingham in the United Kingdom, salt damage investigation and pilot trials in the West Indies associated with airfield pavement damage, and full-scale trials for a new road in Botswana constructed through known saline natural ground.

The laboratory work initiated to examine problems with bituminous surfacing in the West Indies (*9*) indicated that bituminous prime coats were particularly sensitive to damage from small amounts of salts contained in the pavement materials or groundwater, whereas bituminous surface dressings were more resistant to salt damage. The lack of traffic was considered to be a problem that is particularly important for airfield runways and sealed road shoulders.

Field trials in the West Indies and Botswana were constructed to confirm these findings and also to investigate the benefit of several preventive treatments. The details and results of these trials have been reported by Obika and Freer-Hewish (*10*) and Woodbridge et al. (*11*).

Objectives

A design method is proposed to prevent soluble salt damage to thin bituminous road and runway surfacings in warm islands and arid and semiarid inland regions. It is based on laboratory simulation testing of field conditions and field trials.

Factors Influencing Salt Damage

The factors influencing salt damage are climate, geology and hydrogeology, materials characteristics, pavement surfacing design, and construction practice.

Climate

Temperature, relative humidity, wind speed, and rainfall all influence salt damage. They affect evaporation significantly and hence the potential for upward salt migration. Temperature and relative humidity also determine whether salt crystallization thresholds are crossed. These aspects are discussed by Obika et al. (*12*). Precipitation influences the net water balance at a given location and also whether there is a seasonal or perennial moisture deficiency that could provide the conditions for a net upward saline moisture migration. Where rainfall is insufficient to leach out minerals from weathering rocks, in situ accumulation of mineral salts generally occurs.

Geology and Hydrogeology

The depth and quality of groundwater contribute significantly toward creating bituminous surfacing damage from salts. Saline groundwater may result from the solution of minerals present in sediments or from the ingress of seawater to the host material. The predominant type of salt depends on a variety of geochemical processes, the source of the salt, and the local climate. The most commonly encountered salt in many arid and semiarid zones is sodium chloride, known as halite.

In arid and semiarid zones, the capillary moisture rise can be more than 10 m (*13*). The height of the rise depends on a variety of factors, including porosity and temperature gradients.

Materials Characteristics

The various salt types that can contribute to the damage of pavements in dry lands include but are not limited to sodium chloride (NaCl), sodium sulfate (Na_2SO_4), sodium carbonate (Na_2CO_3), and magnesium sulfate ($MgSO_4$).

Fine-grained porous materials can encourage deleterious filamentous crystal growth, and the pore characteristics of the individual particles can influence the movement of saline moisture in the pavement layers. Obika et al. (*12*) discussed the nature and magnitude of salt crystal pressures. For materials of equal mechanical strength, those that contain large pores separated from each other by micropores are the most liable to salt weathering.

Pavement Surfacing Design

The type of bituminous surfacing and its application rate influence the rate of evaporation from the pavement surface and therefore the rate of upward salt migration. Pavement damage from soluble salts appears to be confined to thin bituminous surfaces, generally less than 50 mm thick. However, a few exceptions have been recorded in Algeria (*14*) and Australia (*15*).

In southern Africa, Netterberg et al. (*6*) discovered that damage from sulfates in mine waste pavement material could be prevented by applying a bituminous surface seal that had a ratio of permeability to thickness not exceeding 30 (permeability in millimeters per second, sufacing thickness in millimeters). Thick surfacings minimize evaporation and hence reduce migration and crystallization of salts at the surface.

Obika and Freer-Hewish (*9*) and Woodbridge et al. (*11*) showed that bitumen emulsion primes perform slightly better than bitumen cutback primes in reducing salt damage. The emulsion "sits" on the surface rather than penetrating into the base, thereby forming a less permeable surface than cutback primes. However, emulsion generally gives a poorer bond to the underlying pavement layer.

Construction Practice

The intervals between the construction of the pavement layers—a waterbound or cemented material, a bituminous prime coat, and a final surfacing, such as a surface dressing—can be critical if evaporation is high and if salts are present in the pavement material, the shallow groundwater, or both. Substantial salt accumulation may occur at the exposed surface in periods longer than 24 hr.

Brackish water is often used for compaction or curing of pavement layers. This method can lead to a significant precipitation of salt on the surface of the compacted layer. Also, there is evidence from laboratory studies and field observations to suggest a high risk of surfacing damage when salts in a pavement layer are subjected to repeated wetting and drying (solution and recrystallization).

Risk Evaluation for Bituminous Surfacings in Saline Environments

Salt damage risk evaluation is recommended whenever a thin bituminous surfacing is proposed for a pavement in a warm island or arid or semiarid inland region. Clearly, the damage process is dependent on a complex interaction of many variables, but the proposed design method is based on two significant parameters, salt content and climate, which can be measured relatively easily. Further design parameters can be added as other variables can be linked to the damage process in qualitative terms. The procedure (Figure 1) first allocates values to salt levels in the pavement and subgrade materials, compaction, and, in some situations, groundwater and then allocates values to climatic conditions. The values are combined to provide an overall rating that indicates the damage risk for bituminous surfacings.

Salinity Levels of Materials and Water

Salinity values are required for pavement and subgrade materials, imported and in situ, and compaction and groundwater. Methods for determining salt content are given by Obika and Freer-Hewish (*10*), and it is im-

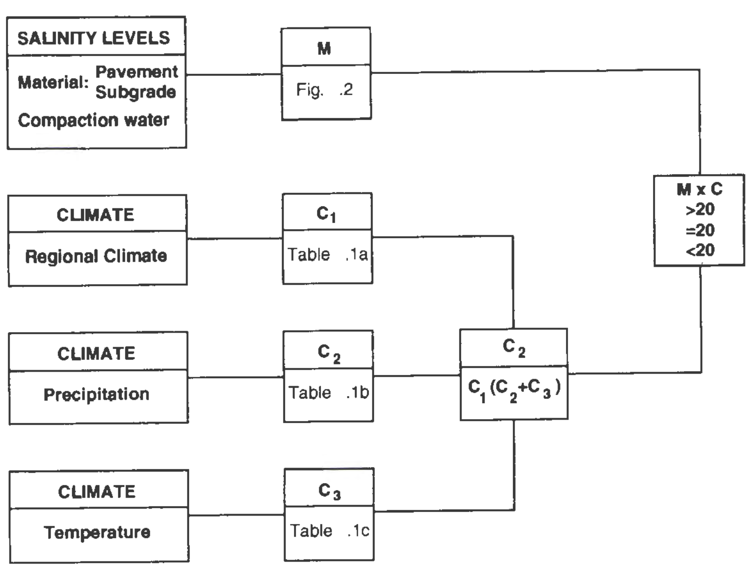


FIGURE 1 Risk analysis for salt damage to bituminous surfacings.

portant to adhere to these methods for consistency and comparability of results.

Salt Measurement

In the first instance, total soluble salts (TSS) will normally be measured; however, it is important to ascertain the dominant salt type for more detailed design and construction control, particularly if salt levels are significant.

Most of the analytical techniques available to determine salt content are time-consuming, and some require a considerable degree of skill. For this reason, methods have been developed whereby the electrical conductivity (EC) of an aqueous solution of the material is measured using a standardized procedure and the values are related to the percentage of TSS of the sample. The absolute relation of EC to salt content is complex, and although it is not possible to obtain a correlation on a global basis, it has been proposed on a regional basis, for example, by Doornkamp et al. (*16*) in the Middle East. The EC is measured in millisiemens per centimeter (a siemen is the reciprocal of the electrical resistance in ohms) at 25°C, and the TSS content is measured in mass percentage.

The correlation between the rapid EC and the TSS measurements for the Botswana field trials, where the TSS was measured according to the method given in BS 1377: Part 3 (*17*), was $TSS = 0.04 + 0.16EC$. A correlation coefficient of 0.9 was obtained for this relationship. All determinations were carried out on the minus 20-mm fraction of the samples, corresponding to about 75 percent by mass of the borrow pit material; it is generally recognized that the major proportion of the salt is contained in the fines.

Materials and Water Rating

Using the appropriate salt levels for materials and water, the weighting value M is determined from Figure 2. An M value of 10 should be adopted if the pavement or subgrade salinity exceeds 0.8 percent TSS regardless of compaction water salinity.

The salt content value used should be the maximum value obtained from the pavement or subgrade materials and may be measured in terms of either TSS or EC if calibrated locally for the materials used. Ideally, the salt content at the surface before bituminous surfacing should be used.

Climatic Conditions

Characteristics of the regional climate, seasonal precipitation pattern, and seasonal temperature pattern are required.

Climate Rating

The project site can be classified regionally as extremely arid to others in Table 1. A value (C_1) appropriate to the regional climate is assigned. Similarly, Tables 1b and c are used to assign values (C_2 and C_3) appropriate to the seasonal precipitation and temperature variations. The overall rating for the climate (C) is obtained by multiplying the sum of C_2 and C_3 by C_1.

Combined Risk Evaluation ($M \times C$)

The combined risk value is obtained by multiplying M by C. For MC values greater than 20, special design and construction measures may be required in order to prevent damage to bituminous surfacings. High MC values relate directly to high risk of salt damage. If the MC value is marginal, it is recommended that a detailed inquiry of the history and performance of existing thin surfacings be implemented. A visual inspection of existing surfacings, particularly for lifting and loss of adhesion between the bituminous surfacing and the underlying layer, is also desirable.

Design Procedures for $MC > 20$

Types of Bituminous Surfacing

For reasons discussed earlier, only surfacings less than 50 mm thick appear to have been damaged by salts, and the degree and rate of damage vary according to the type of bituminous surfacing.

Selection of Bituminous Primes and Primer Seals

The prime surfacings are the most susceptible to damage from salt crystallization, primarily because they are the thinnest and the least effective in reducing evaporation from the underlying pavement. Damage can occur within 2 days of application.

Type and Application Rate

Primes made from penetration-grade bitumen cutback with a highly volatile fluid such as kerosene are more susceptible to salt damage than primes made from an emulsion. Although emulsion is useful to alleviate the onset of salt damage, it can create a tacky surface that cannot be trafficked before final surfacing unless dusted with fine aggregate. Increasing the prime application rate and providing a thicker barrier to reduce evapo-

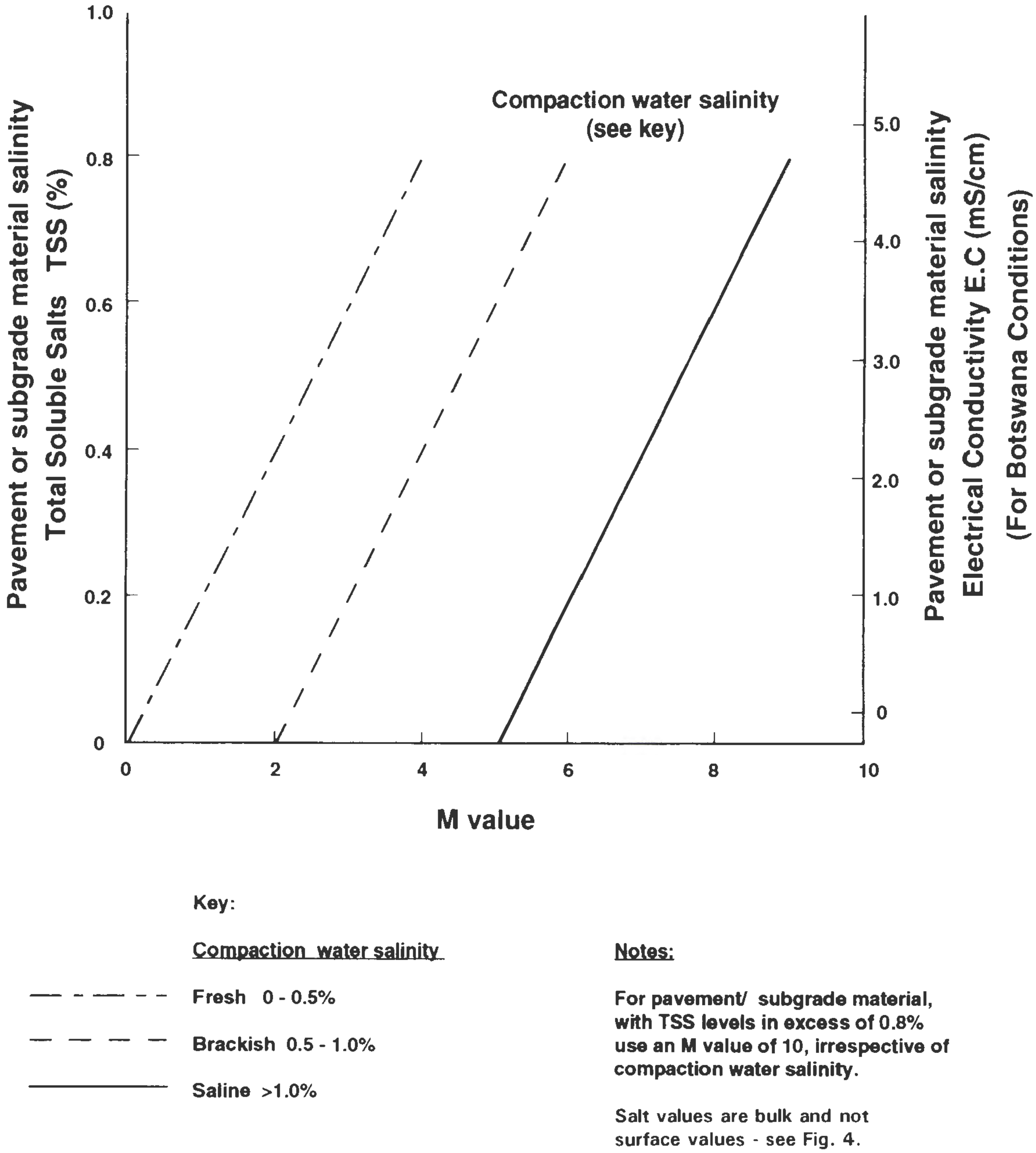


FIGURE 2 Materials risk rating: salt damage to bituminous surfacings.

ration from the pavement may also create a tacky surface.

The salt thresholds shown in Figure 3 provide guidelines for the use of either cutback or emulsion primes. The salt values in Figure 3 refer to the surface (0 to 50 mm) of the pavement just before sealing, and Figure 4 provides a relationship, obtained from the Botswana field trials, between initial salt content within the pavement material at construction and salt content at certain time intervals after construction. Ideally, trials on site to check this relationship are recommended. The values given in Figure 2 for initial risk assessment are initial bulk salt values.

Intervals Between Bituminous Surfacing Pavement Layers

From the foregoing discussion, it is clear that the time intervals between compaction of the road base and the prime and the surface dressing are important. Ideally, primes should be covered immediately if salts are present in the pavement.

TABLE 1 Salt Damage: Risk Evaluation for Climate

a)	Regional Climate:		C_1 Value
	Extremely arid		4
	Arid		4
	Semi-arid		6
	Island coastal		5
	Others		0
b)	Precipitation		C_2 Value
	No marked season of precipitation		2
	Summer precipitation		3
	Winter precipitation		1
c)	Temperature	Range	C_3 Value
	Zone T1	>30°C	3
	Zone T2	20-30°C	2
	Zone T3	10-20°C	1

When the pavement materials have a negligible salt content and there is the possibility of ingress of salt from the water table or subgrade, or both, through capillary action, vulnerable primes and primer seals should be covered within a week by a more substantial surfacing. Actual rates of moisture rise from the Botswana field trials appeared to be of the order of 5 mm/day (*10*). Figure 3 incorporates the time constraints for various conditions. Control of salt movement is another option and is considered below.

Selection of Permanent Surfacings

There is no evidence to indicate salt damage to thick surfacings (> 50 mm), and it is reasonable to assume that no precautions are required for these surfaces. Damage to thin permanent bituminous surfacings takes considerably longer to develop than damage to primes. This period can vary from one week to several years and may depend on the condition of the prime when covered by the permanent surfacing, the type and position of harmful salts in or below the pavement, and trafficking of the surface.

The importance of the impermeability of the bituminous surfacing as a means of retarding the upward rise of salt was mentioned earlier. Surface dressings appear to be impermeable; however, upward movement of moisture has occurred on some of the Botswana field trial sections. Moisture rise beneath impervious pavements has been noted before by Tomlinson (*18*) and Horta (*14*). Cracking of a surface caused by shrinkage, oxidation, or traffic would encourage localized evaporation and salt crystallization.

The salt content thresholds and time intervals between surfacing operations are shown in Figure 3. These values have been designed for protection of primes and are too conservative for the performance of single and double surface dressing seals, but, at present, precise salt-level thresholds for long-term performance of final surfacings have not been substantiated. The maximum salt content thresholds recommended for Botswana are shown in Table 2; the trials are still being monitored for longer-term performance.

The Botswana trials and other damage sites, however, highlight the importance of identifying whether the surfacing will be trafficked or untrafficked. Sealed shoulders and large portions of runway pavement areas are examples of the latter.

These trials indicate that stricter limits are required for untrafficked surfaces. The kneading action of traffic on surfacings appears to be very important in preventing damage and increases the resistance of surface dressings to salt damage. The road trials in Botswana showed detachment of single sealed shoulders alongside the intact double-seal trafficked carriageway.

Control of Salt Movement

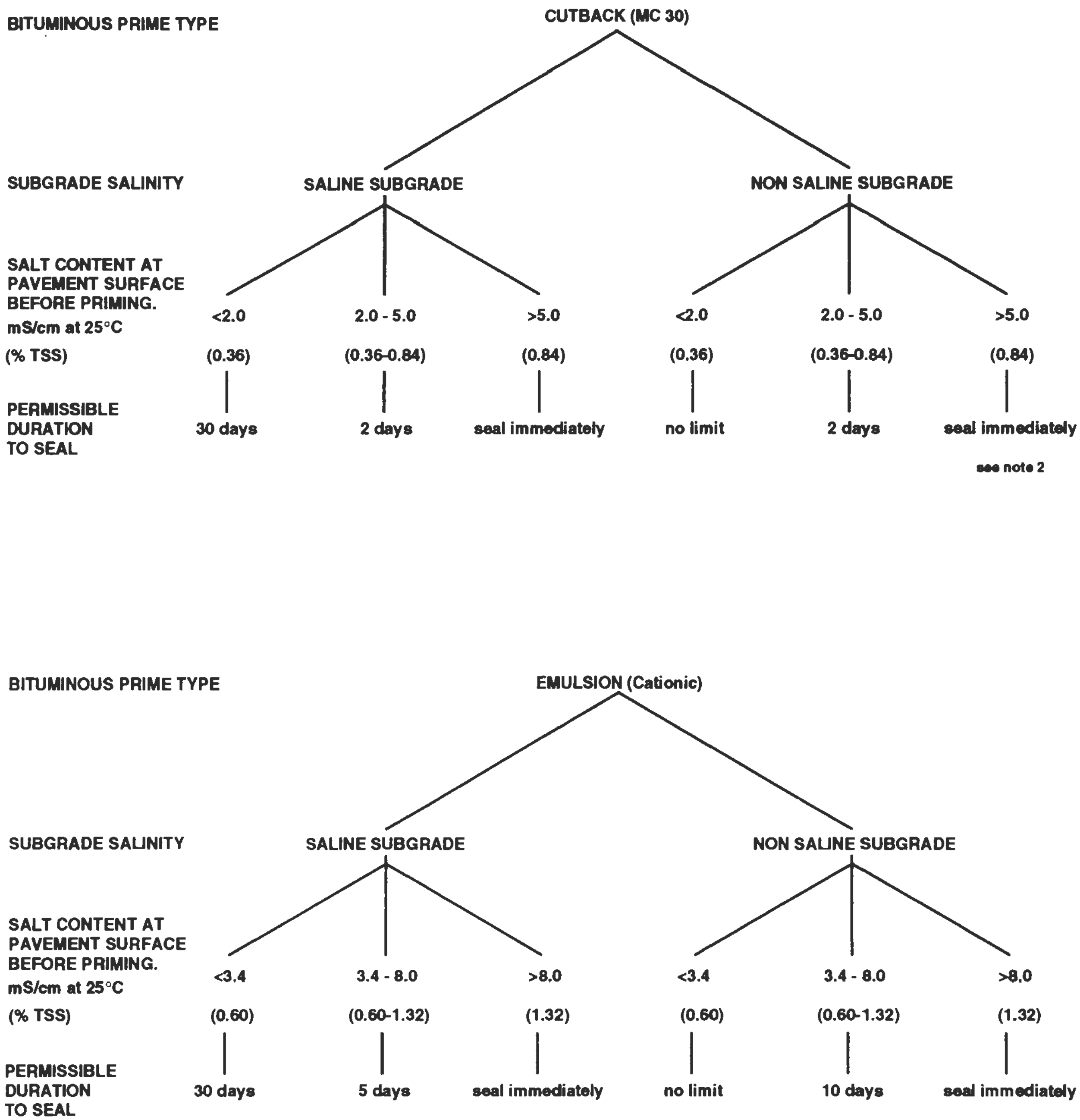
When salts are inherent in the subgrade or groundwater, an impermeable plastic fabric can be introduced at the subgrade-pavement interface. This method was found to be effective in preventing the rise of saline water to the pavement surface and thus preventing surface damage, even though adjacent sections were damaged. A thick bitumen layer placed in the same position was not successful in preventing damage. In Botswana thin bituminous surfacings constructed on saline subgrades without an impermeable plastic fabric were damaged by salts.

Repair of Damaged Surfacing

Prime Surfaces

Damage detected in its early stages can be arrested by rolling, which may control further blistering until more layers can be added and adhesion with the underlying base can be regained.

For severe damage, rolling will not be successful and the surface must be broomed to remove the prime. When the underlying base is still sound, a new spray technique following the guidelines above can be used, but if the base consists of soft aggregates, brooming can



NOTES:

1 Total of time delays between base compaction and sealing should not exceed 30 days. This is the estimated time taken for salts from the saline subgrade to migrate through a 150mm base. Thicker pavements may permit longer delays to seal.

2 Longer duration to seal may be possible if the base surface remains dry with moisture contents typically below 6%

3 A factor of safety of 2 should be applied to salt contents.

FIGURE 3 Permissible intervals between prime and final surfacing in relation to subgrade salinity and pavement surface salinity.

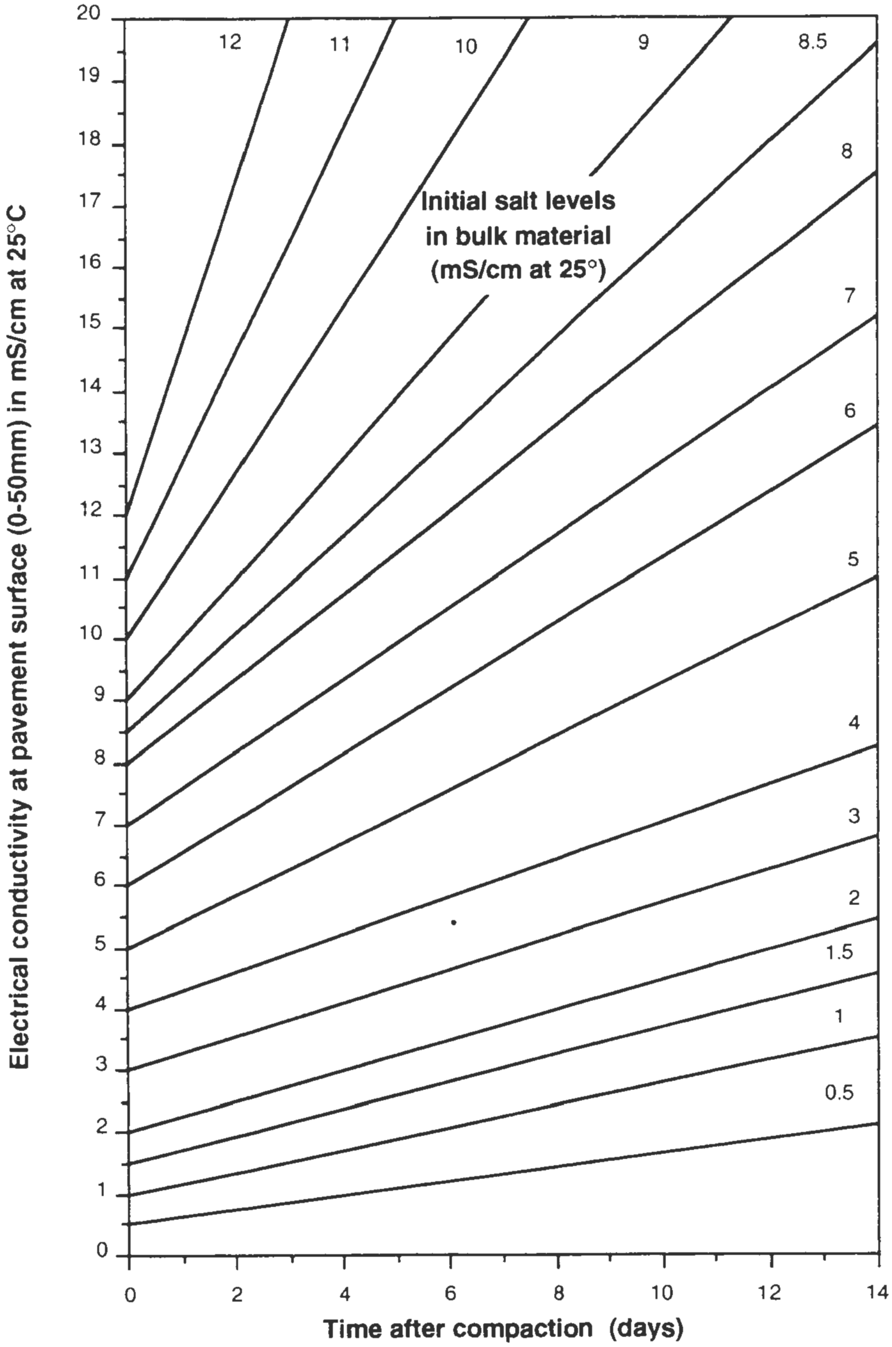


FIGURE 4 Electrical conductivity readings at surface of a layer with time for initial conductivity levels of bulk material for Botswana trial site.

damage the base surface. It may then be difficult to regain the same surface level and quality without scarifying to at least 100 mm.

Final Bituminous Surfacings

Small failures should be treated locally by removing the surface and hand spraying a new surfacing, possibly replacing cutback bitumens with emulsions and increasing the application rate without risking severe bleeding.

Conclusions

In warm arid and semiarid climates, where evaporation exceeds precipitation, soluble salts accumulate in the

TABLE 2 Suggested Minimum Salt Limits for Botswana Project

Surface Type	Traffic Status	Subgrade Condition	Maximum total soluble salt content at surface prior to priming % TSS (0-50 mm sample depth)	
			Emulsion Prime	Cutback Prime
Prime			0.30*	0.20*
Single Surface	Untrafficked	Saline	0.30	0.15
		Non Saline	-	-
"	Trafficked	Saline	0.90	0.70
	"	Non Saline	>1.20	>0.80
Double Surface Treatment	Trafficked		>2.0	>1.0

Notes:

1. *A factor of safety of 2 has been applied.*
2. *For the Botswana calcrete TSS % = 0.04 + 0.16 x Electrical Conductivity (E.C.).*
3. *Salts can be inherent in pavement materials or introduced with brackish/saline compaction water.*
4. *If initial salt contents only are known, obtain an estimate of surface salt content for the appropriate time delay using Fig. 4.*

* *Increased if construction process speeded up to satisfy Fig. 3.*

upper layers of the road pavement and can damage thin bituminous surfacings such as prime coats and surface dressings. The results of laboratory and field studies have identified the importance of climatic factors such as humidity and temperature, intervals between the construction of each pavement layer, surfacing types, and trafficking. The design procedure shows that single values of salt limits, as suggested in other reports, are not appropriate for all surfacing types and construction procedures.

A procedure for risk evaluation of potential salt damage has been developed based on the laboratory and field trials. Risk ratings are assessed for materials, compaction water and groundwater, and climatic conditions for different surfacing types.

Bituminous prime coats are very sensitive to salt damage and can be damaged if the soluble salt content exceeds 0.3 percent TSS in the roadbase material as a whole. Cutback prime is more sensitive to damage than emulsion prime.

Surface dressings are more resistant than bitumen primes to salt damage because of their increased bitumen thickness. The Botswana trial indicated that trafficked single and double surface dressings would not be damaged by roadbase TSS contents up to 0.5 and 1.0 percent, respectively. Trafficking appears to increase the resistance of surface dressing to salt damage. Surface-dressed road shoulders and large areas of runway are especially vulnerable to salt attack at lower salt content levels.

For salt levels at the upper acceptable limits, a prime coat should preferably be excluded, and trials would be advisable to determine the effectiveness of bonding the surface dressing directly onto the roadbase. Alternately, the prime coat could be surface dressed within 2 days of application, but this may be impractical in contract situations.

When traffic is withdrawn from hitherto sound sections, the surfacing becomes damaged.

Methods of preventing the upward rise of salts in solution were incorporated into the trials in Botswana, and an impermeable fabric (plastic) placed at the bottom of the roadbase prevented the upward rise of salt and protected the bituminous surfacing from salt damage without compromising road performance. A thick bitumen layer placed in the same position was not successful in this respect. The technique of a cutoff membrane has been applied to the new road in Botswana.

Remedial treatments for salt damage consist of rolling at early signs of damage followed by further surfacing immediately or removal of damaged material and replacement to the requirements of the design method described in this paper.

ACKNOWLEDGMENTS

The research was carried out by the School of Civil Engineering, University of Birmingham, in collaboration with the Overseas Centre of the U.K. Transport Research Laboratory for the Overseas Development Administration and the Ministry of Works, Transportation and Communications, Botswana. The latter two organizations funded the work. Ove Arup and Partners, Botswana, and Wallace Evans and Partners, U.K., also supported the work described in the paper.

REFERENCES

1. Netterberg, F., and L. J. Maton. Soluble Salt and pH Determinations on Highway Materials. Presented at 6th Regional Conference for Africa on Soil Mechanics and Foundation Engineering, Durban, 1975.
2. Meigs, P. *World Distribution of Arid and Semi-Arid Homoclimates*. Arid Zone Hydrology. UNESCO, 1953, pp. 203–209.
3. Obika, B., R. J. Freer-Hewish, and P. J. Fookes. Soluble Salt Damage to Thin Bituminous Road and Runway Surfaces. *Quarterly Journal of Engineering Geology*, Vol. 22, 1989, pp. 59–73.
4. Weinert, H. H., and M. A. Clauss. Soluble Salts in Road Foundation. In *Proc., 4th Regional Conference for Africa on Soil Mechanics and Foundation Engineering*, Cape Town, 1967, pp. 213–218.

5. Fookes, P. G., and W. J. French. Soluble Salt Damage to Surfaced Roads in the Middle East. *Journal of the Institution of Highway Engineers*, Vol. 24, No. 12, 1977.
6. Netterberg, F., G. E. Blight, P. F. Theron, and G. P. Marais. Salt Damage to Roads with Bases of Crusher-Run Witwatersrand Quartzite. In *Proc., 2nd Conference on Asphalt Pavements for Southern Africa*, Durban, 1974, pp. 34–53.
7. Netterberg, F. *Salt Damage to Roads: An Interim Guide to its Diagnosis, Prevention and Repair*. Institution of Municipal Engineers of South Africa, NITRR, CSIR, 1979, p. 4.
8. Cole, D. C. H., and J. G. Lewis. Progress Report on the Effect of Soluble Salts on Stability of Compacted Soils. In *Proc., 3rd Australia-New Zealand Conference on Soil Mechanics and Foundation Engineering*, Sidney, 1960, pp. 29–31.
9. Obika, B., and R. J. Freer-Hewish. *Salt Damage to Bituminous Surfaces for Highways and Airfield Pavements*. Final report. Overseas Development Administration, University of Birmingham, 1988.
10. Obika, B., and R. J. Freer-Hewish. *The Control of Soluble Salt Damage to Bituminous Road Surfaces in Tropical Environments*. Final report. Overseas Unit, Transport and Road Research Laboratory, University of Birmingham, 1991.
11. Woodbridge, M., B. Obika, D. Newill, and R. J. Freer-Hewish. *Proc., 6th Conference on Asphalt Pavements for Southern Africa*, Cape Town, 1994.
12. Obika, B., R. J. Freer-Hewish, and D. Newill. Physico-Chemical Aspects of Soluble Salt Damage to Thin Bituminous Road Surfacing. Presented at International Conference on the Influence of Ground Chemistry in Construction, IGCC '92, Bristol University, 1992.
13. Cooke, R. U., D. Brunsden, J. C. Doornkamp, and D. K. C. Jones. *Urban Geomorphology in Dry Lands*. Oxford University Press, 1982.
14. Horta, J. C. de O. S. Salt Heaving in the Sahara. *Geotechnique*, Vol. 35, No. 3, 1985, pp. 329–337.
15. Januszke, R. M., and E. H. S. Booth. Soluble Salt Damage to Sprayed Seals on the Stuart Highway. In *Proc., 12th Australian Road Research Board Conference* (ARRB), Part 3, 1984, pp. 18–30.
16. Doornkamp, J. C., and H. A. M. Ibrahim. Electrical Conductivity and Saline Concentrations in Arid Land Groundwaters. *Quarterly Journal of Engineering Geology*, Vol. 19, 1986, pp. 249–250.
17. *British Standard Methods of Test for Soils for Civil Engineering Purposes, Part 4: Compaction-Related Tests*. British Standards Institution, 1990.
18. Tomlinson, M. J. Engineering Problems Associated with Ground Conditions in the Middle East. *Quarterly Journal of Engineering Geology*, Vol. 2, 1978.

Improving Bitumen-Stabilized Mixtures

Anne Valkonen and Pertti Nieminen, *Tampere University of Technology, Finland*

The main purpose of this laboratory study was to research additives that could improve the early strength and water resistance of mixtures stabilized with bitumen emulsion. The study was performed on two materials: till with high fines content and recycled base-course material. Stabilization techniques have developed over the years, but some problems such as slow strengthening and low water resistance have occurred. In this laboratory study, it was found that a small amount of portland cement improves the early strength and water resistance. The early strength of mixtures treated with portland cement was 1.5 to 5.3 times higher compared with mixtures without additives. Also, water resistance was improved considerably when portland cement was added. Cementlike additives such as ground blast furnace slag also improved the early strength, whereas $Ca(OH)_2$ and gypsum did not. Additives also had an effect on the dry density of specimens when constant compaction energy was used. The bitumen-stabilized mixture is water susceptible, and in autumn, additives are often needed. One possible additive is portland cement; laboratory tests indicate very promising results with this additive. Further information and research are needed before the dimensioning of layers can be carried out using these kinds of mixtures.

The major problems of Finnish low-volume roads are surface cracking and deformation because of uneven bearing capacity and frost damage. Recycling of layer materials is a cost-effective and energy-efficient method of road reconstruction. Layer materials of low-volume roads are quite suitable for bitumen stabilization. In Finland the first bitumen stabilization was carried out in 1985. The stabilization technique and machinery have developed since those days. Field performance has been generally good, but, because of a lack of knowledge of mixture properties and design methods, some problems have occurred. Especially when stabilization is used in autumn, bitumen-stabilized layers possess some relatively unfavorable characteristics such as slow development of strength, low water susceptibility, and strength at early ages.

In Finland instructions for design and analytical dimensioning of bitumen-stabilized layers were published in 1994 (*1*). Further research is needed, however, before the mixture properties can be used as parameters in analytical dimensioning of layers.

Materials and Methods

In this laboratory study, two different kinds of aggregate were stabilized: till with high fines content, and recycled base-course material with low fines content. According to the natrium hydroxide test, both aggregates had a low content of organic matter. The gradation curves are shown in Figure 1. The binder was cationic bitumen emulsion with a medium setting rate. (The distillation residue was 65 percent.) The additives tested were ordinary portland cement (OPC), $Ca(OH)_2$,

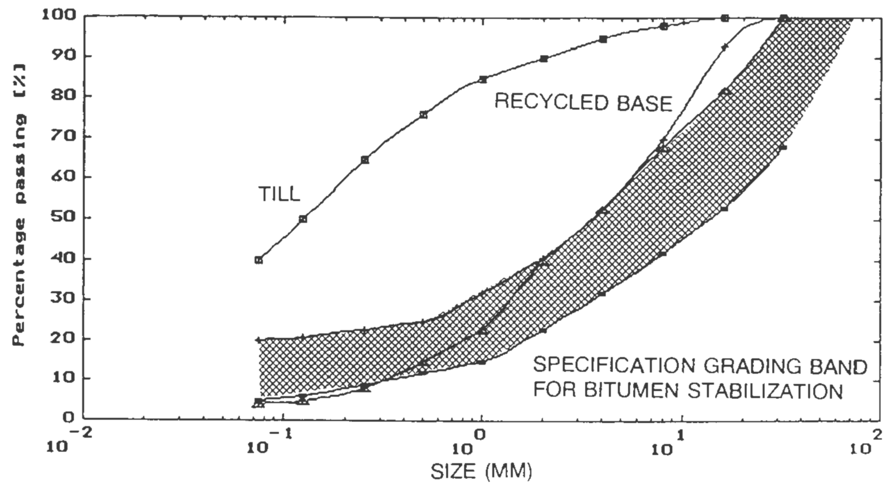


FIGURE 1 Gradation of till, recycled base course aggregate, and Finnish specification grading band for bitumen stabilization.

ground blast furnace slag (GBFS)(Blaine 400 m^2/kg), and gypsum (a power plant byproduct). The stabilized till material mixtures (1, 2, 3, and 4) and recycled aggregate mixtures (5, 6, 7, and 8) are shown in Table 1. All the proportions are calculated by weight of dry aggregate.

The moisture content of till material mixtures was 0.8 percent optimum moisture content while compacted. The moisture content of the recycled base course material was 2 percent in field conditions before reconstruction. This percentage was also the mixing water content used in laboratory tests. The total moisture content of the recycled layer mixtures was 4.2 percent. The water was added to the room-dried aggregate and mixed thoroughly. Bitumen emulsion was combined with the cold and wet aggregate by using a mechanical mixer. All the mixture specimens were made with a gyratory compacting machine. The till material mixtures were compacted by using constant compaction energy for each specimen. For that reason, there are differences in dry densities of specimens (Table 1, Mixes 1–4). The recycled base-course material mixtures were compacted until the fixed density of the specimen was reached (Table 1, Mixes 5–8). Curing was carried out at room temperature. The specimens were wrapped in plastic so that evaporation was possible from the top surface of the cylinder. All the specimens were characterized by an indirect tensile test at room temperature (22°C). The water susceptibility of till material mixtures was evaluated by a capillary water soak test for one day. After water soaking, the specimens were characterized by an indirect tensile test. The indirect tensile strength (ITS) loss was calculated as follows:

$$\text{Strength loss (\%)} = \frac{\text{ITS (unsoaked specimens)}}{\text{ITS (soaked specimens)}} * 100 \qquad (1)$$

TABLE 1 Mixture Design and Average Dry Density of Specimens

	till material				recycled base coarse aggregate			
mixture	mix1	mix2	mix3	mix4	mix5	mix6	mix7	mix8
water (%)	6	4.8	4.8	4.8	4.2	4.2	4.2	4.2
residue asphalt (%)		5	5	5	4	4	4	4
OPC (%)			1			1		
GBFS (%)							2	
$Ca(OH)^2$ (%)				1				
Gypsum (%)								3
Average dry density of specimens (kN/m^3)	21.7	20.9	20.4	19.6	22.0	22.0	22.0	22.0

Analysis of Results

The reference mixture (Mix 1, Table 1) gains strength without binder because of moisture evaporation, which improves the cohesion among fine particles. Because of their high water susceptibility, reference specimens could not resist the water damage. The till material with high fines content was suitable for bitumen stabilization, but the strength loss after the water soaking test was 80 percent (Figure 2). The effect of ordinary portland cement on indirect tensile strength was apparent only at early curing ages and after water soaking. The strength loss was only 40 percent after the water soak-

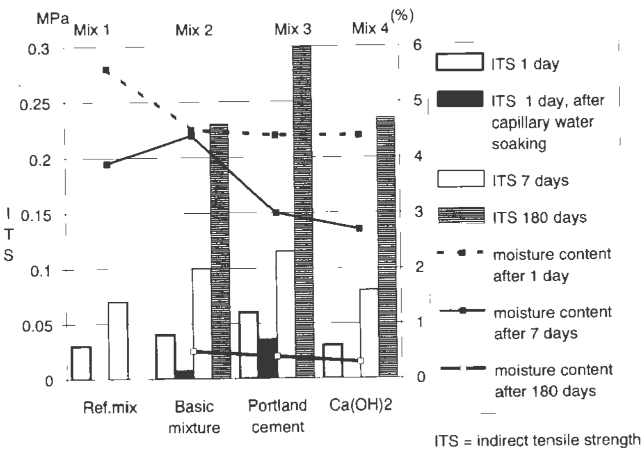


FIGURE 2 Indirect tensile strength and moisture content after 1, 7, and 180 days of curing.

ing test. The portland-cement-treated and one-day-cured unsoaked specimens had 1.5 times higher ITS than the specimens without additives (Mix 3, Table 1, and Figure 2). For the specimens cured 7 days, the differences in strength of mixtures were equalized. After 180 curing days, the portland-cement-treated mixture had 1.3 times higher ITS than that of the basic mixture (Figure 2). $Ca(OH)_2$ has no effect on strength or water susceptibility. This result may be a consequence of the low dry density of the specimens.

Moisture evaporation of the specimens was researched and is shown in Figures 2 and 3. Curing decreased the amount of moisture retained in the mixture. The increase in strength is related to the rate of moisture reduction.

Additives used with till material [OPC and $Ca(OH)_2$] decreased the density of specimens (Mixes 3 and 4, Table 1). Reduction of density was more significant when the additive was $Ca(OH)_2$ compared with ordinary

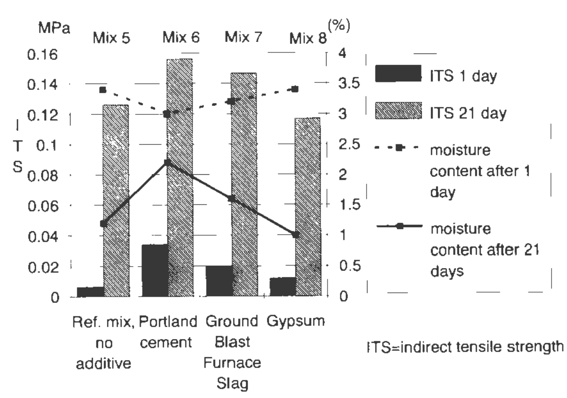


FIGURE 3 Indirect tensile strength and moisture content after 1 and 21 days of curing.

portland cement. This density reduction is caused by hydrophilic additives, which adsorb part of the moisture needed during compaction. When OPC was added, the density decreased by 0.5 kN/m^3 compared with the density of the bitumen-stabilized mixture. The reduction of density was 1.3 kN/m^3 when $Ca(OH)_2$ was added.

Four different mixtures were based on recycled aggregate (Mixes 5–8, Table 1, and Figure 3). One of the mixtures was a reference mixture without additives. The effect of OPC was significant at early ages as shown in Figure 3. The portland-cement-treated mixture had 5.3 times higher ITS than that of the reference mixture. GBFS also had an effect on indirect tensile strength after a one-day curing period. In this GBFS-added mixture (Figure 3), the ITS was 3.1 times higher than that of the mixture without additives. The gypsum could not improve ITS as well as OPC or GBFS. The strength gain was only 1.8 times higher than that of the basic mixture at early curing ages. After a 21-day curing period, no improvement was observed in the gypsum-treated specimen. The differences among mixtures characterized by ITS were equalized when the curing time was 21 days. The ITS of OPC- and GBFS-treated mixtures was only 1.2 to 1.3 times higher compared with that of the basic mixture without additive (Figure 3). The effect of additives seems to be negligible when the curing time is 21 days.

Conclusions

Curing increases the ITS and decreases the total moisture content. Ordinary portland cement improves the ITS more than ground blast furnace slag, $Ca(OH)_2$, or gypsum. This improvement is significant only at early curing ages. After a one-day curing period, portland-cement-treated unsoaked specimens had 1.5 to 5.3 times higher indirect tensile strength than that of the bitumen-stabilized mixtures without additives. After a longer curing period (7, 21, and 181 days), the indirect tensile strength was only 1.2 to 1.3 times higher than that of the reference mixtures.

The water soaking test results indicated an improvement in water resistance when portland cement was added. When 1 percent ordinary portland cement was mixed with the bitumen-stabilized aggregate, the strength gain (after water soaking) was 4.5 times higher than that of the mixture without cement. The optimum water content when compacted is usually higher than the ideal water content for bitumen stabilization. For that reason, a compromise concerning total moisture content must be accepted. In this study, it was found that when compaction is carried out using constant compaction energy, the bitumen-emulsion-stabilized specimens had 0.8 kN/m^3 lower dry density compared with that of till specimens compacted at optimum moisture content. The portland-cement-treated mixture has 1.3 kN/m^3 lower dry density compared with that of till aggregate without any binders. Calcium hydroxide caused the most significant reduction (2.1 kN/m^3) to the dry density.

In this study, the curing was conducted at room temperature, and moisture evaporation was allowed. The amount of moisture (1 to 2 percent) after a 21-day curing is likely to be slightly too low when long-period field conditions are duplicated.

Laboratory research results indicate the importance of moisture loss in the stabilized mixture. Another important matter is the compactibility of the mixture. In many cases, it is perhaps unnecessary to add portland cement to the mixtures if the weather is dry after stabilization and the stabilized layer is allowed to dry. If the base-course reconstruction must be done in autumn when the weather in Finland is changeable and wet, one should consider the use of cementlike additives in bitumen-stabilized mixtures.

REFERENCE

1. *Bitumistabilointi* (*Instructions for Bitumen Stabilization*). Finnish National Road Administration, Helsinki, 1994.

Granulated Blast Furnace Slag in Base Course of Low-Volume Roads

Kari Pylkkänen, *Tampere University of Technology, Finland*

The use of granulated blast furnace slag (GBFS) mixed with an old base and wearing course aggregate on a low-volume gravel road is presented. GBFS is a poorly graded by-product of the iron industry and has latent hydraulic properties. The purpose of using this product was to increase the bearing capacity in spring and decrease frost heave differences. In preliminary tests, the compressive strength of compacted specimens increased as a function of GBFS content (10, 30, and 50 percent). The rate of strength gain was slow; considerable development was not noted until after 91 days of curing. In the test section the aggregate consisted of 30 percent lime-activated GBFS and 70 percent mixed wearing and base course aggregate. All control test specimens showed increased strength with increased curing time. The rate of strength gain was slow and affected by the density of the specimens and curing conditions. Decreased density or curing temperature (field conditions) as well as water immersion before testing decreased the compressive and tensile strengths. Once up to ultimate load, stressed specimens were recompressed after a predetermined curing time. In many cases, the strength was on the same or an even higher level than in the first compression. The strength was influenced by the recovering time, the water content of the specimen, and the recuring temperature. Both in spring and autumn the total surface deflection in FWD measurements was 0.2 mm smaller on the GBFS-aggregate section than on the reference section with normal aggregate layers. The difference in maximum and minimum frost heave was on the same level before and after construction.

Road 3424 is situated in central Finland and has an average daily traffic (ADT) of 590 including heavy vehicles transporting timber to the nearby wood-processing industry. The wearing course is made of gravel and there are sections with low bearing capacity, especially during the thawing period. The road also has uneven frost heave. Road 3424 is one among seven low-volume roads in a project to examine different ways of reducing the afore-mentioned problems. The methods applied were strengthening the base course by stabilizing it with hydraulic binders based on industrial by-products or by using geogrids, improving the drainage with gravel and vertical or horizontal drains, and preventing frost penetration by using thermal insulation. On Road 3424, two sections were stabilized using either activated granulated blast furnace slag (GBFS) or hydraulic binder based on desulfurization waste. In one section, geogrid was used, and one section was isolated with expanded clay pellets. In this paper, only the results of the use of GBFS are presented.

Materials and Methods

The grain size distributions of the aggregate and GBFS are presented in Figure 1. The aggregate of the test section is a well-graded mixture of old wearing (crushed rock) and base course aggregates. Compared with the grading envelope for wearing course aggregate for

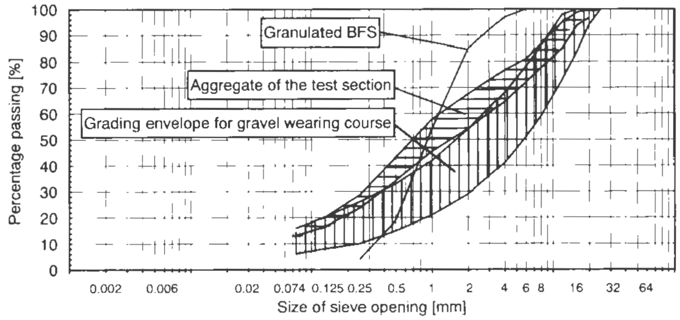


FIGURE 1 Grading of aggregates.

gravel roads, the aggregate mixture was somewhat too sandy. When determined using 3 percent NaOH solution, the amount of organic impurities in the aggregate was high. The grading of the GBFS is typical for granulated slags; it is poorly graded with small fines content. According to the chemical composition, the basicity determined as the ratio between $CaO + Al_2O_3 + MgO$ and SiO of the used GBFS was 1.6. In many specifications the basicity is used as a factor defining the hydraulic activity: increased basicity means increased activity (*1*). The hydraulics also increases the finer and more glassy the slag is. When attention is paid to the glass content (90 percent) and basicity, the used slag can be considered active with hydraulic properties, but when used without grinding, slow-strength gain and low strength level are expected.

Test specimens with a diameter of 100 mm and height of 115 mm were compacted using a gyratory compactor, sealed in plastic bags, and cured at 60 percent relative humidity and 22°C. The loading rate in compression was < 0.14 MPa/sec.

Surface deflections of the test sections were measured using a Kuabfalling weight deflectometer and applying an impulse load of 5000 kg. The deflections were measured at a distance of 0, 0.2, 0.45, 0.6, 0.9, and 1.2 m from the loading center.

Preliminary Tests

Tests Without Activator

In the tests without activator, the GBFS content was 10, 30, and 50 percent of the total amount of aggregate. The aggregate-GBFS mixture was compacted with 6 percent excess water to a wet density of 22.3 kN/m^3. The water content of GBFS was not taken into account; therefore, the dry density of the specimens decreased with increased GBFS content (Table 1). Since compressive strength is a function of dry density (*1*), it should lead to decreased strength. The compressive strength did, however, increase when the GBFS content increased (and density decreased). Because of the slow strength progress, this result is most clearly seen after a long curing time.

The addition of poorly graded granulated slag to well-graded aggregate reduces the fines content and impedes the compactibility by increasing the sand content. In this study a six-times-higher compactive effort was needed to achieve the desired wet density when GBFS content was increased from 30 to 50 percent. Increasing the content from 10 to 30 percent tripled the needed compactive effort. With 10 percent GBFS content the compactibility was about the same as without slag. When constant compactive effort (20 revolutions) was used, the aggregate was compacted about 7 percent denser without GBFS than with 30 percent of GBFS.

Tests with Unslaked Lime Activator

The rapid hydration and strength gain of GBFS require the use of alkali activators. In blended cements, $Ca(OH)_2$, produced in the hydration of portland cement, serves as an activator (*2*). In France the unground GBFS is generally activated using lime (*3*). In this study, unslaked lime (CaO) was used to accelerate the hydration and remove water from GBFS. The aggregate mixture contained 30 percent GBFS, and the effects of lime

TABLE 1 Results of Preliminary Tests

GBFS - content [%]	Compaction effort [number of revolutions]	Dry Density [kN/m^3]	Compressive Strength [kPa]				
			0 d	7 d	28 d	91 d	154 d
10	10	21.0	60	90	100	150	140
30	35	20.7	70	100	140	190	350
50	222	20.5	90	100	160	280	670

TABLE 2 Test Results with Activated GBFS

CaO content [% of GBFS]	CaO mixing moment	Dry Density [kN/m^3]	Compaction effort [number of revolutions]	Compressive Strength [kPa] 7 d	28 d	91 d
1		20.8	60	90	90	120
3		20.7	43	80	100	120
	premixed	20.8	-	120	190	240
10		20.9	115	560	580	740
	premixed	20.7	23	250	290	370
		21.4	396	810	840	770

content, lime mixing moment, and the specimen density on the compressive strength were studied. According to the results presented in Table 2, the activation effect was observed only with a content of 10 percent. Compressive strengths were then improved considerably. When the lime was added to GBFS one week before it was mixed with aggregate, the activation effect was decreased to half the activation effect with simultaneous but separate mixing.

Construction

Short (50-m) test sections were constructed in August 1992. A schematic cross section of the structure is seen in Figure 2. The old wearing course and part of the layer below it were loosened and premixed with a backhoe loader and grader. About 100 kg/m^2 of GBFS (activated two weeks before with 2 percent of unslaked lime) was spread with a horizontal sand spreader in two stages. After each stage, the layers were mixed by driving a spring tooth harrow three or four times over them. The layers were compacted with a backhoe loader and loaded lorries. The upper layers of crushed rock aggregate were constructed immediately after compaction to prevent drying of the stabilized layer.

Control Tests

Tests with Specimens

The control specimens were compacted using the aggregate mixed in the test section. According to the sieve analysis, the aggregate was equivalent to the 70/30 percent mixture used in the preliminary laboratory tests. The control tests included studies of the effect of specimen density, curing conditions, and water immersion on the compressive and tensile strength. The results (Figure 3) again prove the remarkable influence of density, not only on the achieved strength level but also on the rate of strength gain. Between 28 to 91 and 91 to 182 days, it was two times faster with dense than with loose specimens. Also, the unfavorable effect (low curing temperature) of the late construction date (August) is noted when laboratory- and field-cured specimens are compared. During the first 1.5 months, the average daily air temperature near the test section fluctuated between 8 and 17°C. After 1.5 months of construction, the temperature decreased permanently to ≤ 5°C, causing retarded hydration reactions and stopping the strength development.

Immersing the specimens for one day in water before testing also had a harmful influence on the compressive strength. Immersion doubled the water content of the specimens to 9 percent and decreased the strength to 40 percent of the nonimmersed specimens. The relation of the water content to compressive strength can also be examined using the results from the specimens compressed when as dry as possible. Even though these specimens were older when compressed, they were cured between 182 and 270 days in unfavorable conditions for hydration reactions so that the greater strength of the "dry" specimens is obviously not due to hydraulic strength gain.

The compression up to ultimate load-stressed specimens was repeated once or twice after the first com-

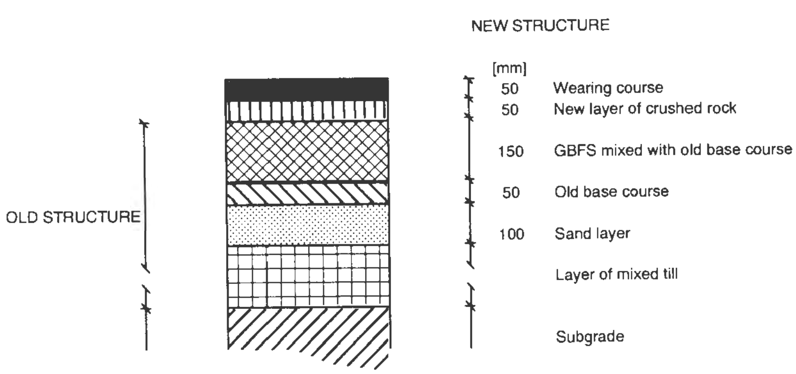


FIGURE 2 Compression results of control specimens.

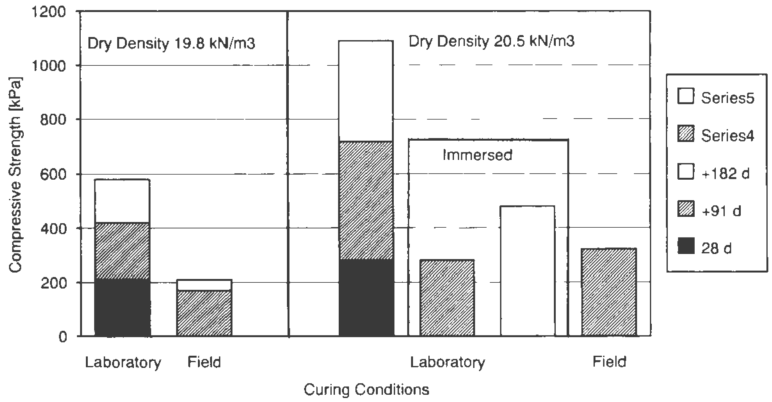


FIGURE 3 Strength after recuring.

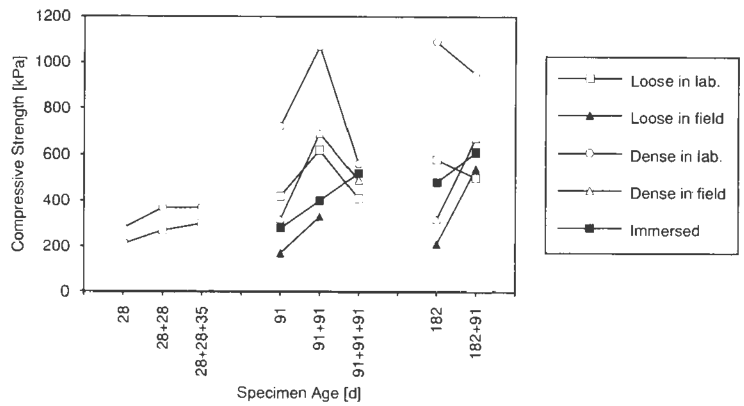


FIGURE 4 Cross section of structure.

pression. The recuring conditions were maintained the same as before the first compression and the recuring time varied from 1 to 3 months depending on the first compression age. Only the curing temperature of the field-cured specimens (recured in the laboratory) and the moisture content of the immersed specimens were different. The aim of this recompression procedure was to see if the GBFS-aggregate mixture had any self-cementing competence after the first or second failure.

The results (Figure 4) are interesting. If the curing conditions were unchanged, the strength in the second compression could still increase to a higher level during the recuring time when the first breakdown happened at a fairly young age (28 or 91 days). Changing the curing conditions to conditions more favorable for hydration reactions, that is, increasing the recuring temperature or water content of the specimen, led to the same results even with specimens compressed for the first time after a half year of curing. In unchanged recuring conditions, the strength of these specimens decreased. In some cases, the specimens were also compressed after the second breakdown. The strength of the younger (28 + 28 days) specimens was increased at least to the same level as in the previous compression. After the second compression, the strength of the older (91 + 91 days) specimens was increased only when the specimens were immersed before the first compression.

Tensile strength was determined by a splitting tensile test. The results (Table 3) indicate the same kind of in-

TABLE 3 Tensile Strength of Control Specimens

Dry Density [kN/m³]	Cured	Splitting Tensile Strength [kPa]							
		Age of the specimen [d]							
		28	91	182	28+28	28+28+35	91+91	91+91+91	182+91
19.8	L	11		67	17	18			46
	F		9	12			24	28	23
20.5	L	17		119	25	26			95
	LI		30	70			39	37	80
	F		21	19			70	67	52

L = in laboratory, F = in the field, LI = in the laboratory; immersed before test

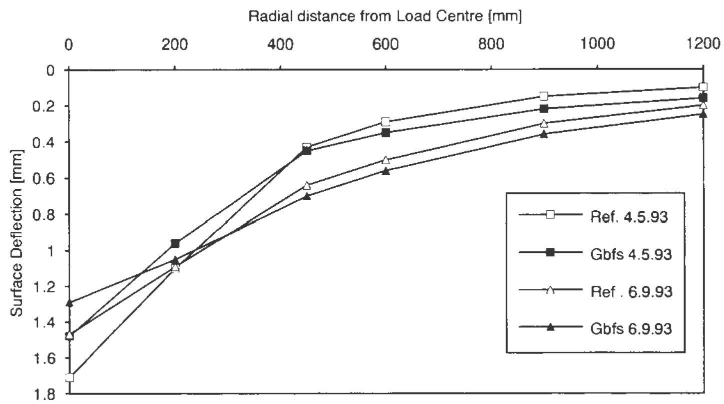


FIGURE 5 Deflection curves according to FWD measurements.

creasing development of strength as a function of curing time as in compressive strength. The ratio of tensile and compressive strength was about 0.06 for young specimens and increased to 0.12 for older specimens. A reducing effect of decreased curing temperature, lower dry density, and water immersing was also seen in tensile test results. Some recovering after the first failure was also noticed.

Falling Weight Deflectometer

The surface deflections of the FWD measurements in May and September are presented in Figure 5. In the load center, the difference between spring and autumn deflections is about 0.2 mm in both sections. In both seasons, the total deflection of the GBFS section is, however, 0.2 mm smaller than on the reference section. The difference is mainly formed between the first and the second deflections. In spring, the surface curvature index (SCI) (4) is 0.52 for the GBFS section and 0.61 for the reference section. In autumn, SCI is, respectively, 0.24 and 0.42.

Frost Heave

Frost heave of the test sections was measured twice before construction and once after construction. Before construction, the difference in the maximum and minimum frost heave of the GBFS section was 5 to 11 cm (Figure 6). The next year, after stabilization, the difference was still 8 cm even though the total frost heave was smaller than before construction. Decreased total frost heave was also observed on the reference section, so it is mainly due to milder climatic conditions in the winter of 1993 than in the winter before construction.

Discussion of Results

The increasing compressive and tensile strengths of compacted specimens in preliminary as well as in con-

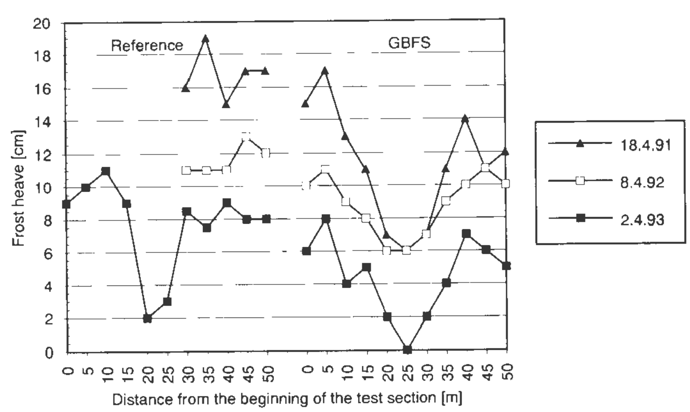


FIGURE 6 Frost heave before and after construction.

trol tests indicate that, mixed with ordinary wearing and base course aggregate of a low-volume road, unground GBFS works as a hydraulic binder and results in slow strength gain even though the aggregate contains organic impurities. Concerning bearing capacity, the hardening property of GBFS compensates for the negative effects on grading of mixing this poorly graded by-product with well-graded aggregate. Mixing can, however, have a positive influence, for example, on the drainage of the base layer, when the aggregate gradation becomes poorer and the permeability increases. Because of a decreased fines content, the amount of water the layer is able to absorb also decreases, which reduces bearing problems caused by excess water. GBFS can also be used to proportion an aggregate with open-graded to well-graded aggregate with good compactibility, which leads to a dense layer. In Finland this effect has been used successfully in some crushed rock aggregates on two test roads.

The self-healing property after the first and sometimes even the second failure was interesting and makes one think that it should be used in road construction. However, this property requires more research to be understood properly. One advantage of this self-healing property is that the strengthening of structure is not as susceptible to the disturbing effects of site traffic as normally stabilized structures.

The constructed layer of mixed GBFS-aggregate material is now only $1^1/_2$ years old, so its effects on bearing capacity and frost heave cannot yet be properly evaluated. FWD measurements indicate, however, that the mixed GBFS-aggregate layer is working as a semirigid layer decreasing the total deflection of the road surface compared to structures built with unstabilized aggregate. The GBFS stabilization has, however, not been able to decrease the difference between maximum and minimum frost heave.

REFERENCES

1. Cowell, M. J., and L. H. Irwin. Effects of Compaction Delays and Multiple Treatments on the Strength of Cement Stabilized Soil. In *Transportation Research Record 702*, TRB, National Research Council, Washington, D.C., 1979, pp. 191–198.
2. Sersale, R. Advances in Portland and Blended Cements. In *Proc., 9th International Congress on the Chemistry of Cement*, Vol. I: Congress Reports. National Council for Cement and Building Materials, New Delhi, India, 1992, pp. 261–303.
3. Ballie, M., and M. Barberet. Emploi du laitier à haute teneur en fines pour le traitement des assises. *Bulletin liaison Laboratoire des Ponts et Chaussées*, Vol. 101, May-June 1979, pp. 53–59.
4. Majidzadeh, K. An Overview of Deflection Parameters for Performance Analysis. In *Proc., International Symposium on Bearing Capacity of Road and Airfields*, Norwegian Institute of Technology, Trondheim, 1982, pp. 22–30.

Granular Base Failures in Low-Volume Roads in Ontario, Canada

C. A. Rogers, S. I. Szoke, and R. G. Gorman, *Ontario Ministry of Transportation, Canada*

In 1990, 74 percent of the 196 773 km of Ontario roads was low-volume roads, and 68.5 percent of all roads was unpaved. For the same year in Canada, there was 825 743 km of roads, and 65 percent was unpaved and generally classed as low volume. This large percentage of low-volume roads is a reflection of the size of Canada and its sparse population. Gravel roads may fail due to a number of mechanisms. In this paper, only those cases are considered in which the road was constructed with aggregates that met or were predicted to meet the specifications but failed either because of unforeseen changes in the materials or because of unrecognized contaminants in the materials, which still met the specification. These cases can be divided into failures because of low permeability caused by the presence of plastic or micaceous fines, failures because of low permeability caused by the presence of bacteria and algae, and failures because of breakdown of the coarse aggregate to sand sizes. The Ministry of Transportation has recently changed the specifications for granular base course aggregate to introduce a Micro-Deval test in place of petrographic examination requirements and the Los Angeles abrasion and impact test. A maximum of 25 percent loss is permitted for granular base and 30 percent for granular subbase. In addition, there are strict grading requirements and a requirement that the materials be nonplastic and that there be less than 10 percent mica on the 75-μm sieve.

Road construction started in Ontario over 200 years ago in 1783 when the first United Empire Loyalists arrived to take up their land grants. The forerunner of the early road was the portage or "carrying place," which was a trail around rapids or between two waterways. These portage routes, developed by the aboriginal peoples of Canada, became the early road corridors. With the settlers came the wagon, creating an immediate need for passable roads. In 1783 the first 25 km of road, running west from Kingston to Bath in eastern Ontario, was built. This early road was a blazed trail through the dense forest. Because of the difficult terrain, the early roads were only usable during the dry midsummer months and during the winter when hard-packed snow provided a smooth traveling surface. By 1874 the length of major colonization roads was 1534 km. During the mid-1800s, a Canadian invention, the plank road, became popular. Plank roads were constructed by securing planks of wood with spikes to longitudinal timbers in the road bed and covering the planks with soil or sand and hot pitch to reduce friction and prolong the life of the road (*1*). The availability of good lumber meant that one kilometer of plank road could be constructed at one-fourth the cost of a kilometer of macadamized road. At the same time, the government focused on establishing the railroad system. This focus left Ontario's roads to deteriorate at an alarming rate. The Ontario Good Roads Association

TABLE 1 Ontario's Road System Length by Surface Type and Length in Kilometers and Low-Volume Road (AADT <2,000) Percentage, 1990

Road Type	Provincial Highways[a] (0 <AADT <15,000+)	Municipal Roads[a] (0 <AADT <4,000)	Access Roads[b] (0 <AADT <400)	System Total
Paved	17,379	44,610	0	61,989
Surface Treated	3,289	24,278	0	27,567
Gravel	998	62,797	39,646	103,441
Earth	0	3,752	0	3,752
Other	0	24	0	24
Length	21,666	135,461	39,646	196,773
AADT <2000	11,047 (51%)	94,823 (70%)	39,646 (100%)	145,516 (74%)

Source: [a] Ministry of Transportation, 1993
[b] Ministry of Natural Resources, 1993

was formed in 1894 by municipal officials and citizens in a concerted effort to protect their interests and make others aware of the benefits of well-made roads. Organized road building in Ontario began in 1896 when an engineer, A. W. Campbell, was appointed Provincial Instructor in Road Making.

In 1916 the Provincial Department of Public Highways was created under a Minister of Public Works and Highways. In 1916 there was approximately 88 000 km of country roads, mostly in southern Ontario. In the early 1920s, settlements in the northern regions of Ontario were still almost isolated, linked only by a few crude roads, most transport being by water or rail. Road construction in northern Ontario was instrumental in the development of the mining and lumbering potential of these vast areas. Strong highway links to the southern markets for northern resources and products ensured the prosperity of northern communities; many were still dependent on resource-based economies. After the Second World War, road construction and infrastructure development accelerated in both southern and northern Ontario. Ontario became Canada's most populated and affluent province with a population reaching 10 million in 1990 and enjoying one of the best road systems in the world. This road system includes 21 666 km of provincial highways, 135 461 km of municipal roads, and 39 646 km of access roads (Tables 1 and 2). There are about fifty people for each km of road in Ontario. Municipal roads and access roads provide the network among smaller communities and serve as connecting links for industry and commerce to main highways and export-import opportunities to Canada and the United States. The Ministry of Transportation (MTO) and the Ministries of Northern Development and Mines and Natural Resources are the provincial agencies responsible for new construction and maintenance of Ontario's road system. Approximate annual provincial government expenditures are $1 billion for provincial highways, $750 million for municipal roads, and $10 million for access roads.

Ontario provincial road design guidelines indicate that secondary highways with an annual average daily traffic (AADT) of less than 2,000 fall within the low-volume road design criteria. Secondary highways are further subdivided into three categories. Major secondary highways, with AADT greater than 1,000, serve

TABLE 2 Ontario's Road System Length by Program Type and Length in Kilometers, 1990

Program Type	Length (km)
Provincial Highways Program[a]	
King's	15,797
Secondary	5,663
Tertiary	206
Subtotal	21,666
Municipal Roads Program[a]	
Upper Tier (counties and regions)	20,917
Large Lower Tier (cities, towns, townships)	41,951
Small Lower Tier (towns, villages, townships, Indian reserves)	63,065
Unincorporated Areas	9,528
Subtotal	135,461
Access Road Program[b]	
Forest Access	28,135
Agreement Forest	2,763
Private Forest	1,196
Fuelwood	31
Ministry Service	1,169
Recreation Access	3,750
Cottage Access	549
Residential Subdivision	34
Public Transport	333
Winter	433
Mining Access	418
Other	835
Subtotal	39,646
Total	196,773

Source: [a] Ministry of Transportation, 1993
[b] Ministry of Natural Resources, 1993

major regional centers, communities, and resource areas. Intermediate secondary highways, with AADT of 400 to 1,000, link villages, recreational areas, and resource areas. Minor secondary highways, with AADT of less than 400, provide access to recreational and resource areas or parallel a higher-level provincial road. In 1990, 74 percent of the 196 773 km of Ontario roads was low-volume roads, 68.5 percent was unpaved. For the same year in Canada, there was 825 743 km of roads, and 65 percent was unpaved (2). This large proportion of low-volume roads is a reflection of the size of Canada and its sparse population.

A persistent challenge in any type of road construction in Ontario has been the need to find suitable construction aggregates that will consistently meet stringent performance requirements in a geographic area where severe climatic changes prevail, geological history is complex, and the age of soils and rocks ranges from less than 10,000 years to over 3.5 billion years. Aggregates for road base can be obtained from glacially derived sand and gravel deposits or from a wide variety of bedrock formations.

Case Histories of Granular Base Failures

Performance of Granular Base

The function of granular base in road construction is to carry the load of vehicles and to drain water from the pavement structure (Figure 1). In cold climates it also acts as an insulating layer, preventing or reducing the penetration of frost into the underlying, sometimes frost-susceptible, finer-grained soils. To accomplish this, suitable granular bases must be strong, free draining, and durable.

Some of these properties can be measured by performance tests. Strength of compacted granular base is measured by the California bearing ratio (CBR). Typical values for granular base are over 100 percent. Typical values for loose, sandy granular base are between 20 and 60 percent. A very loose sandy granular, such as dune sand, in which a wheeled vehicle might get stuck, would have values less than 15 percent. Strength of the particles making up the coarse portion can also be measured in the Los Angeles abrasion and impact test. Stability of the base, measured by CBR, is related to the grading of the material and to the degree of interparticle friction. For practical purposes, interparticle friction is related to the amount of freshly fractured faces on the coarse aggregate particles.

The drainage capability can be measured by permeability tests (ASTM D 2434). Satisfactory granular bases will typically have permeabilities between 10^{-4} and 10^{-3}cm/sec. If permeability of a granular base is less than 10^{-4}cm/sec (typically in the range of 10^{-5} to 10^{-7} cm/sec or less), failure will probably occur. Permeability is largely determined by the amount of material finer than 75 μm in granular base. As fines increase above about 8 percent by mass, permeability is rapidly reduced. In Ontario a maximum amount of 8 percent passing the 75 μm sieve is permitted in base crushed from gravel. In base crushed from bedrock, up to 10 percent fines is permitted because the fines from bedrock are usually not as claylike as those from gravel. Occasionally, base materials that meet and exceed the grading requirement will fail in service. In wet weather, failure may occur during placement of the pavement or shortly after having placed the overlying asphalt pavement. Failure is usually seen as severe rutting of the granular base or alligator cracking of the overlying asphalt within a few hours or days of placement. The reason for the rapid failure of the asphalt is the very poor compaction achieved against the underlying saturated and spongy base. If the base is placed under relatively dry conditions, failure may not show up for some months or years. Paved roads will show failure by severe to moderate depressions in the wheel paths and alligator cracking of the overlying asphalt pavement.

Gravel roads may fail because of a number of mechanisms. In this paper (Figure 2), only those cases are considered in which the road was constructed with aggregates that met or were predicted to meet the specifications but failed either because of unforeseen changes in the materials or because of unrecognized contaminants in the materials, which still met the specification (Table 3). These cases can be divided into failures because of low permeability caused by the presence of plastic or micaceous fines, failures because of low permeability caused by the presence of bacteria and algae, and failures because of breakdown of the coarse aggregate to sand sizes.

Failures Due to Low Permeability Caused by Plastic Fines

Moosonee Area

Moosonee, located in the Hudson Bay Lowlands of northeastern Ontario, is a flat-lying, featureless, swampy plain that slopes gradually toward James Bay. Access is by air, rail, or boat. This community serves as a commercial and institutional center. Gravel-surface, low-volume roads provide local access.

Aggregate materials used in the building of these gravel roads originate from two main sources. First, some of the river bars in the Moose river have been traditional sources of fine to coarse sand and fine gravel.

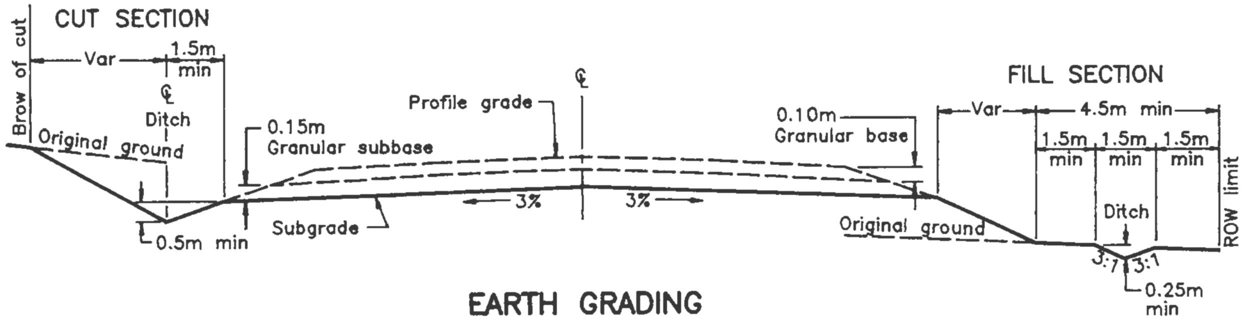


FIGURE 1 Typical Ontario low-volume road.

Second, flat-lying, medium-bedded Middle Devonian limestones are quarried for road gravel. Because of the high water table, all overburden stripping, drilling, blasting, and crushing operations are performed on a demand basis during the winter months with equipment brought in by rail. The quarried rock is an excellent aggregate exceeding all the requirements of granular base and being suitable for use in portland cement concrete (Table 3). The climate in the summer is frequently wet, and the poor condition of the gravel roads has been a constant source of distress to the people. The roads, when wet, become soupy and have ruts up to 150 mm deep. Walking along the road quickly covers the walker's boots with loose gravel of a muddy consistency, not unlike high-slump concrete. The granular base made from the quarried rock exhibits all the characteristics of failure because of the presence of plastic fines. Testing of the quarried rock showed that plastic fines in the pavements were not present. The source of the plastic fines was an unresolved problem.

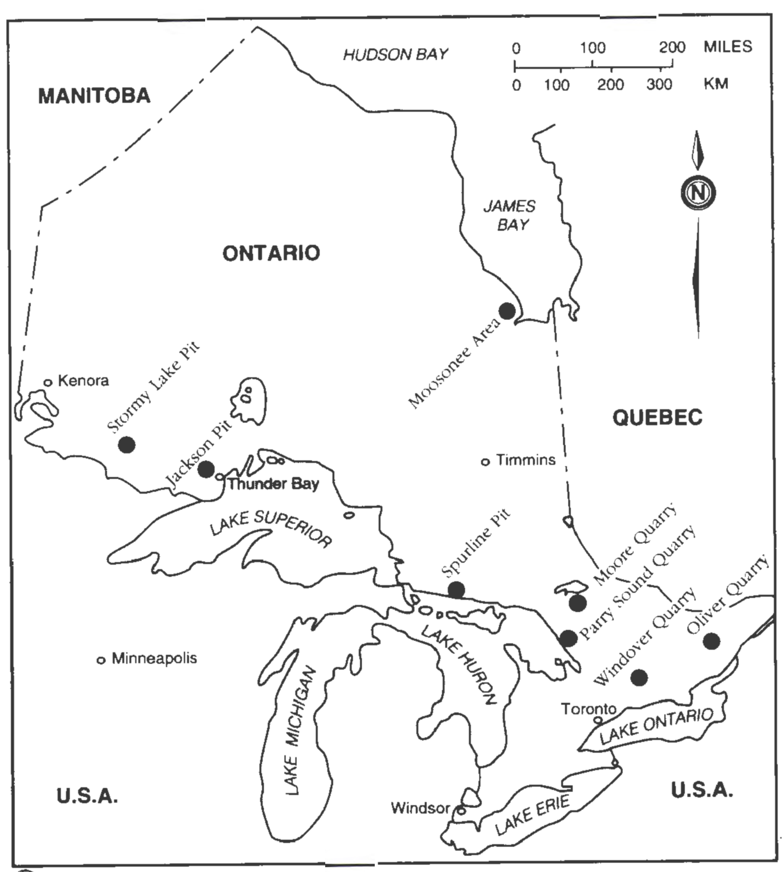


FIGURE 2 Location map.

TABLE 3 Granular Base Test Results (Ranges Provided Where Applicable)

Case Study Location	Petrographic Number Granular	Petrographic Number Concrete	Los Angeles Abrasion, %	MgS04 C.A., %	MgS04 F.A., %	Pass 75 µm, %	Plasticity of Failed Material	Absorption, %
Moosonee Area, River Bar	134-208	196-257	-	-	-	5.0	plastic	1.0-1.7
Moosonee Area, Limestone	100-118	112-147	29	2-9	-	-	plastic	2.1-2.8
Windover Quarry	102	106	24	4	15	-	plastic	-
Moore Quarry	107-133	148-234	34-42	11	22	7.8-8.0	plastic	1.4-1.5
Thunder Bay Area[a]	120-140	160-180	17	-	-	5.0-7.0	plastic	-
Jackson Pit	101-103	118	19	2	-	4.8-6.2	non-plastic	0.57
Stormy Lake Pit	102-140	124-190	23-26	2-5	5-20	2.9-15.9	non-plastic	-
Spurline Pit	100-103	100-112	14-18	0.3-1.3	1.2-5.1	0.5-3.4	plastic	0.40-0.50
Oliver Quarry[b]	202	415	89	17	-	-	non-plastic	1.64
Parry Sound Quarry[c]	121	212	54 (39-60)	5	-	7.6	non-plastic	0.44
Granular A Specification Limits	200	-	60	-	-	2.0-8.0 pit 2.0-10.0 quarry	non-plastic	-

[a]Thunder Bay area coarse and fine aggregate Micro-Deval Abrasion Test results are 15 and 26%, respectively.
[b]Oliver Quarry Coarse Aggregate Micro-Deval Abrasion and Freeze-Thaw Test results are 13 and 3.2%, respectively.
[c]Parry Sound Quarry Coarse and Fine Aggregate Micro-Deval Abrasion Test results are 33 and 11%, respectively.

After an exhaustive search, it was found that the source of the plastic fines was the sands dredged from the Moose River. In order to meet the gradation specified for gravel roads more easily and to reduce cost, sands dredged from the river were blended in small quantities into the crushed quarried rock. These sands were of low fines content and apparently nonplastic. Inspection of stockpiles of the sand showed the presence of armored clay balls up to 100 mm in diameter. The clay was of a highly plastic nature and from the marine clays found over much of the region. Their presence in the sand had not been noticed because the balls were completely enveloped with coarse sand. When the balls were broken open, the highly plastic clay could be seen enclosed by a thin rind of sand. The obvious solution to the problem of the poor roads in Moosonee was to stop blending river sand into the quarried stone.

Windover Quarry

The Windover Quarry is located 15 km north of Buckhorn in Petersborough County. The site is a butte about 30 m high above the surrounding country. The geology of the site consists of about 19 m of horizontally bedded Middle Ordovician limestone interbedded with small amounts of slightly sandy and clayey dolomitic limestone. The limestone butte is an outlier unconformably lying on a peneplane of high-grade metamorphic gneisses of Late Precambrian age. Before the quarry was developed, the butte was drilled with a diamond drill in the center and the core was tested. The results showed that the rock met all the physical requirements for concrete and asphalt use and would certainly be an excellent source of granular base for an adjacent low-volume road (Table 3). There was no evidence of weathering of the rock in the drill core.

During reconstruction of the adjacent road, a quarry face about 18 m high was opened by drilling and blasting at the side of the butte. The contractor crushed the rock with a portable crushing plant. Extensive testing was done for gradation, and the crushed materials met the specification. No testing was done for other properties since previous testing showed the rock to be of excellent quality. During wet weather and before the road was paved with asphalt, some sections of the gravel surface failed and had to be replaced. The failure was similar to that experienced in Moosonee. The roads, when wet, become soupy and have ruts up to 150 mm deep. Walking along the road quickly covers the boots with mud. The granular base had become contaminated by plastic fines.

The source of the plastic fines was found to be two weathered beds, 1 and 2 m thick, of clayey dolomitic limestone in the quarry face. At the site of the drill core, these beds were unweathered, but at the side of the butte, these beds were deeply weathered for a distance of about 5 to 10 m into the face. These weathered beds had altered to a plastic, low-strength rock. The inspection staff and the contractor had ignored, or not considered significant, the deeply weathered nature of the rock since it only made up a small proportion of the face, which was composed of unweathered limestones. These beds of easily weathered dolomitic limestones are well known in this area of Ontario and are called "green markers" by local quarry operators. The rocks get their name from their greenish-grey color caused by the presence of reduced iron associated with the mineral dolomite. After slight weathering, these beds assume a characteristic light-brown color. When encountered in a quarry, they are normally unweathered but are susceptible to frost action, which causes popouts in concrete and asphalt, so they are not used for road bases (*3*). Crushed rock containing these unweathered green markers is usually satisfactory for use as granular base. The lessons learned from this experience were the necessity always to test the properties of material during production, whatever the previous exploratory testing has indicated, and also to remember that preliminary samples taken with a diamond drill may not reveal potential problems elsewhere on the site.

Moore Quarry

The Moore Quarry is located on the south shore of Lake Nipissing. The quarry was tested and shown in contract documents as meeting the specifications for use as a source of granular base aggregates for the reconstruction of a local low-volume road (Table 3).

The geology of the quarry is complex. The quarry face is in the side of an upthrust block of a vertical fault. The rock is highly fractured and consists of metamorphosed migmatite and gneissic biotite granite of Late Precambrian age. On the downthrust block of the fault a veneer of thin-bedded Middle Ordovician shaley dolostone overlies the gneiss.

The quarry was highly unusual because the operator did not have to drill and blast the rock before crushing. The contractor was able to extract the rock from the fault zone by digging it out with a large mechanical shovel. This was no doubt possible because of the sheared and broken nature of the rocks adjacent to the fault. The rock was crushed in a portable crushing plant. Extensive gradation testing was done, and the crushed materials met the specification. Before the road was paved with asphalt, some sections of the surface failed and had to be replaced. The failure was due to the presence of plastic fines, which made the granular material impermeable to water. The source of the plastic fines was highly plastic red clay found in the fault zone. In addition, several highly weathered, very soft biotite mica seams are associated with the fault. The red clay and mica were responsible for reducing the permeability of the compacted crushed rock, leading to rutting and soft spots on the grade. The contractor had found that as his operation moved away from the fault, it became more difficult to pull down rock with his excavator. As a result, he moved his operation along the strike of the fault in a search for more easily extracted rock. It was here that he encountered the plastic clay and mica seams. An interesting observation was the formation of miniature slope failures on the sides of the stockpile of contaminated granular base. These miniature debris slides could be initiated by pouring small amounts of water on the material. The lessons to be learned from this experience were to distrust any quarry operation that does not require drilling and blasting and to conduct an extensive investigation before approval of new sources.

Thunder Bay Area

Thunder Bay is on the northwest shore of Lake Superior near Jackson Pit (Figure 2). In this area, there has been extensive deposition of gravels by glacio-lacustrine action in the past 10,000 years. The area is noted for extensive raised beach deposits about 30 m above the present lake level. These beaches were formed when the level of Lake Superior was raised by glacial ice blocking the present outlet. The beach deposits are noted for their coarse gravels and cobbles with the relative absence of finer gravels and sand.

Following beach formation, the level of the glacial lake rose above the beach level. There was a period of deposition of highly plastic clay (Plasticity Index, 16 to 20) on the upper surface of the beaches. Following subsequent lowering of lake levels, the plastic clays were washed by precipitation into the openwork beach gravels so that today the upper layers of the raised beaches are clean and apparently suitable sources of aggregate. At depth (>1 m), however, the upper surfaces of cobbles and gravels are covered with a coating of the highly plastic clay. During exploration, testing of the gravels did not reveal the presence of significant amounts of the clay.

The current test for plasticity of gravels calls for testing the fraction finer than 425 μm. The fraction between 425 and 75 μm is made up of good-quality sand. The gritty nature of the sand can mask the plastic nature of the fines. Consequently, gravels from two sources were produced that met the grading (<8 percent passing the 75-μm sieve) and plasticity requirements. When used in highway construction, the overlying asphalt pavement failed within a few hours of placement. The failures appeared as severe alligator cracking and distortion in the wheel paths. The underlying granular base was found to be saturated and impermeable ($<10^{-6}$ cm/sec). The cause of the impermeability was clay derived from the clay-coated gravel particles.

These failures were extremely costly, in one case costing more than \$1.5 million. As a result, MTO is considering changing the test method for plasticity of granular base. The testing should be done on the fraction passing 75 μm, rather than the fraction passing 425 μm.

Failures Due to Low Permeability Caused by Mica

Jackson Pit

Jackson Pit is located at the southeast corner of the Kaministikwia River Bridge on Highway 102 west of Thunder Bay. The crushed gravel and sand from this source was tested and shown in contract documents as meeting the specification requirements for granular base and for asphalt paving. The source was opened and used for road construction.

Bedrock underlying the Jackson Pit and its vicinity is over 2.5 billion years old. The bedrock and gravels of this area are composed of extensively altered and metamorphosed mafic metavolcanic metasedimentary and felsic intrusive rocks. Glaciofluvial outwash gravel is found in Jackson Pit. This material was deposited along the course of the ancient Kaministikwia River meltwater channel that emptied into the Lake Superior basin. The outwash had its origins at the Dog Lake Moraine 17 km north of the pit. Outwash gravel deposits near Jackson Pit are veneered by glacial lake red clay. In places the clay exceeds depths of 10 m and may also sometimes be found as boulders associated with the outwash gravels.

It was found during asphalt paving that the granular base was unstable; the asphalt pavement moved under a person's weight and was judged to have a deflection of about 5 mm. After the first day of paving, the westbound lane showed alligator cracking and broke up for a length of 260 m. The base was found to be saturated and to have low stability. The asphalt pavement was removed from this area and the grade allowed to dry before repaving. Several other shorter sections broke up in the same manner. A longer section on the eastbound lane also broke up and dished very badly. The asphalt pavement was removed, the base excavated to a depth of 150 mm, and a 16-mm crushed stone was used to replace the base before repaving. After the first winter, two further areas of pavement break-up developed, each about 5 to 7 m long. There were also several random longitudinal cracks and general distortion throughout the job, probably because of frost heaving.

The failed material had a fines content of 5.4 to 7.0 percent, which is well within the maximum allowed -75 μm content of 8 percent for crushed gravel. The stockpiled material when tested had 4.0 percent -75-μm content, with a standard deviation of 0.31 percent ($n = 31$). It was initially thought that clay boulders associated with the outwash deposit were responsible for the granular base failure. The granular base fines, however, proved to be nonplastic. The aggregate met and exceeded all other physical requirements for granular base. Petrographic examination and X-ray diffraction showed the 150-μm to -75-μm sieve fractions to consist of about 27 percent chlorite and weathered chlorite schist. This mica was thought to be responsible for the failure. The CBR was 122 percent, but the permeability was so low that it was judged "relatively impermeable" (probably less than 10^{-5} cm/sec). The major source of the chlorite schist was found to be bedrock outcropping just to the north of the source. The schist was found abundantly as weathered cobbles and boulders in the gravel source. Crushing of the gravel concentrated the weak and friable chlorite schist in the fine aggregate.

The effect of mica on permeability results from its large surface area per unit mass, a result of its flat flakey particle shape. For instance, a flake of mica with a ratio of thickness to size of 1:10 has four times the surface area of a spherical quartz particle of the same mass. In addition to the effect of large surface area, mica is extremely efficient at blocking the flow of water through porous media because the flakes orient themselves perpendicular to the direction of flow. This ability is used in drill mud technology to stop or reduce "lost circulation" into permeable bedrock formations below the surface.

Mica in fine aggregate can reduce the permeability of granular base aggregates to the point where failure can occur. As a result of this experience, examination of the 75-μm sieve fraction of all crushed granular base aggregates was adopted in this part of Ontario. If the mica content is greater than about 10 percent, further testing and study are conducted before approval of the material.

Stormy Lake Pit

Stormy Lake Pit No. 3 is located in an unsurveyed area in Kenora District, about 30 km south of Highway 17 on the west side of Highway 622. It is in a small glaciofluvial outwash gravel deposit located between glacial moraines. This source was tested and shown in contract documents as being suitable for use as granular base and asphalt paving.

Bedrock in this area is part of the Wabigoon Subprovince of the Superior Structural Province of the Canadian Shield. The Wabigoon Subprovince is an east-trending granite-greenstone belt containing metavolcanic and subordinate metasedimentary rocks. These Early Precambrian rocks are surrounded and cut by granitoid batholiths.

Granular base produced from the pit met and exceeded all specification requirements and was judged to be well graded, strong, and sound. This aggregate would normally be considered suitable and acceptable for use as granular base. The one qualification of this observation was the presence of 23 percent mica in the 75-μm fraction. Further testing showed that the granular base had a low permeability of 3.7×10^{-5} cm/sec, which revealed lack of a potential permeability problem.

During the Proctor compaction test, there was very substantial breakdown of both stone- and sand-sized material. Stone contents were reduced by some 1.6 times, and the fines increased approximately threefold. Following the Proctor test, permeabilities were on the order of 3×10^{-7}. There was, at the same time, a marked reduction in the mica present in the sand portion, especially a threefold reduction in that retained on the 75-μm sieve. It is likely that the mica had broken down and that most of it was finer than 75 μm. X-ray diffraction carried out on the passing 75-μm material showed that these micas consisted of chlorite and biotite. The X-ray diffraction signature of the Stormy Lake fines was identical to that of the fines found in Jackson Pit. At this point, while the contractor was still crushing in the pit, it was decided to stop production of granular base; 220,000 tons of subbase had already been placed on this 43-km contract. This subbase was unstable, and it was decided to spread a thin layer of the granular base from the Stormy Lake Pit on the subbase to improve stability. At considerable extra cost, Granular A was imported from a pit some 30 km further from the job.

From this study, it was concluded that the total amount of mica is not as important as the amount retained on or passing the 75-μm sieve.

Failures Due to Low Permeability Caused by Algae and Bacteria

MTO encountered a very unusual failure with the granular base used on a contract on Highway 17. During this failure, the granular material became "liquiplastic," and transport trucks actually bogged down. The source of the granular material was a pit located near Algoma Mills on the north shore of Lake Huron. Crushed gravel from this source exceeded all physical requirements for use as granular base, and the material was also used successfully for asphalt paving (Table 3).

The material forming the gravel in the Spurline Pit and vicinity is made up of a complex array of rocks of Middle Precambrian age. Proterozoic rocks of the Huronian Supergroup and post-Huronian diabase intrusions form the bedrock underlying the immediate area. Overlying the Precambrian rocks are glacial and postglacial deposits related to the last major ice advance and melting of a continental ice sheet during the Great Ice Age. Low to moderate topography characterizes most of the area. Swamps and bogs are common in poorly drained areas.

Of particular importance with respect to the Spurline site are the glacial Lake Nipissing beach sands and gravels exposed within the pit. Carbon dating of wood found within the gravels showed an age of 6,500 years before the present. It was from these beach gravels that the granular materials were produced. Within the pit faces, well-developed accretionary beds dipped approximately 20 to 30 degrees toward Lake Huron. Faces within the pit showed well-developed imbricated zones, sand run, infill zones, and thin layers of open work, with fine granules considered as having been deposited at the swash-backwash zone during fair weather conditions. Fairly well sorted, medium-to-coarse sand was exposed in lensitic remnants. Lenses of fairly well sorted, fine-to-coarse subrounded gravel were common. Many of the gravel particles in the open-work zones were encrusted with iron-bearing brown coatings very similar to axle grease in feel, color, and texture.

In conjunction with W. Fyfe, Department of Geology, University of Western Ontario, it was verified that the greasy matrix material that encrusted the beach gravels was a complex, fine-grained product of the activity of an array of iron oxide bacteria. The material was largely composed of organic molecules with dispersed iron oxide, manganese oxide, and possibly clay minerals with particle sizes on the order of 0.01 μm and less. The surface area of these fine matrix materials was astronomical, as was their water-adsorbing capacity. Therefore, drainage through the granulars would be severely restricted.

In this case, it was apparent that certain conditions existed that assisted in the formation of the iron oxide bacteria discovered encrusting the gravels. They include

1. The presence of iron-bearing beds in the Huronian Supergroup (quartzite, feldspathic quartzite, and arkose),
2. A beach environment that contained imbricated open work (high porosity) of fine to coarse gravel clasts, and
3. The presence of a water body (carbon-rich bog or swamp providing organic matter and iron-, aluminum-, and manganese-rich waters, combined with a relatively high water table in the pit).

Iron oxide and algae bacteria have been found in similar deposits throughout the province, but they are uncommon and have not been recognized as a problem in the past. Now, when iron oxide bacteria are recognized, the source is not used for granular base or subbase.

Failures Due to Coarse Aggregate Breakdown

Oliver Quarry

The Oliver Quarry is located 2 km south of Perth in Lanark County. It was opened for local low-volume road construction. The quarry face consists of about 3 m of horizontally bedded orthoquartzitic sandstone of Early Ordovician age. The rock consists of alternating thin beds of hard and friable sandstone. The friable beds are weakly cemented and can be broken to their constituent sand grains by being hit with a hammer.

It is not known if the quarry was tested before being used. Testing results of rock samples taken from the face are shown in Table 3. It can be seen that the Los Angeles abrasion and impact loss is extremely high (89 percent loss). At the time of construction in 1967, there was no requirement in Ontario for granular base aggregates to meet any Los Angeles abrasion and impact loss requirement. The crusted stone met the gradation requirements for granular base use that required a minimum of 50 percent coarser than the 4.75-mm sieve and less than 8 percent passing the 75-μm sieve. Following construction, the road was left open to traffic over the winter before paving the following spring.

In the spring it was found that many of the coarse aggregate particles had broken down to sand-sized particles. This breakdown was caused by the abrasion and impact of vehicle tires on the particles. The pavement was rutted, had low stability, and showed some unevenness. It was found that, despite the breakdown, the pavement drained well. The upper surface of the sandy granular base was graded off into the shoulders, and new granular base was then placed before asphalt paving. As a result of this experience, specifications for maximum Los Angeles abrasion and impact losses of 60 percent for granular base aggregates were introduced in Ontario in 1970.

Parry Sound Quarry

Parry Sound Quarry is located at the intersection of Highways 124 and 69 in the district of Parry Sound. The rock was not tested before it was listed in the contract documents as being satisfactory for use as granular base and hot-mix asphalt aggregate. During blasting of bedrock adjacent to the unopened quarry for associated road construction, the potentially poor quality of the rock was noted, and a drilling and testing investigation was conducted during the winter when construction had ceased.

The bedrock at this location consists of a highly metamorphosed and strongly foliated amphibolite gneiss of Late Precambrian age. The rock has a dark grey color because of the presence of about 50 percent dark-colored amphibole (hornblende). The remainder of the rock consists of quartz and feldspar with small amounts of biotite mica. The degree of interlocking of the minerals is generally low because rounded grain boundaries are frequent. The lack of interlocking texture accounts for the generally weak nature of these rocks, which are thought to have been volcanic in origin, but high-grade metamorphism has destroyed all original minerals and textures.

Petrographic examination showed that the crushed rock was reasonably satisfactory, but particles smaller than 10 mm were often weak and brittle. Examination of the sand-sized portion found that about 25 percent of the particles could be easily broken by impact or rubbing into their constituent minerals, each grain being between 0.5 to 1.5 mm in diameter. The generally brittle nature of the rock was confirmed by the Los Angeles abrasion and impact losses of up to 60 percent (Table 3). The stone was also found to break down in the Proctor compaction test. It was concluded that the rock would make a marginal aggregate for use as granular base. The contractor, however, had already indicated his intention to use the quarry site as his source of aggregate. Directing the contractor to move to another site would have resulted in considerable extra costs. It was decided to use the quarry despite its marginal nature. The aggregate from the quarry performed satisfactorily as granular base, but it was noted that the stone did break down, and this led to some loss of bearing capacity and more rutting and potholing of the base than might normally have been the case. The source was not used for hot-mix asphalt aggregate.

New Aggregate Test Methods

In the early 1980s it was recognized that some of the traditional aggregate quality tests suffered from a number of defects. For instance, the Los Angeles abrasion and impact test, although of satisfactory precision, is done on oven-dried aggregate even though it is recognized that aggregates are always wet in the pavement and water significantly weakens some rock types. This test is useful for measuring the behavior of particles under a steel drum roller and for measuring breakdown during repeated handling and stockpiling, but it is not useful for identifying the majority of aggregates that cause granular base failure.

In 1985 MTO started looking for alternative wet abrasion tests to replace the Los Angeles abrasion and impact test. The test showing most promise was the Micro-Deval abrasion test (4), which was a development of the Deval abrasion test first used in the 1870s. The Deval abrasion test is still described in the AASHTO Manual (AASHTO T 4-35) but has been largely forgotten because of a very poor correlation with field performance. The Micro-Deval test is a wet abrasion and grinding test done in a 5-L steel jar containing 5000 g of 9.5-mm steel balls, 2 L of water, and a 1500-g test sample of 19- to 9.5-mm aggregate. The jar is turned at 100 rpm for 2 hr, and the amount of breakdown of the aggregate is measured by sieving the sample over a 1.18-mm sieve. Weak, water-susceptible aggregates break down rather easily and give relatively high losses. The precision of the test is good. For instance, for an average 12 percent loss in the test, the multilaboratory coefficient of variation is 6.4 percent of the test value, or, in other words, different laboratories will report values between 10.9 and 13.1 percent 19 times in 20. In France the test has been used for a number of years for evaluating granular base aggregates. More recently, it has been adopted in Quebec for selecting base course aggregates from limestone quarries of the St. Lawrence Lowlands.

The research methodology was to select about 100 coarse aggregates of known field performance from across Ontario. These aggregates were then tested in a variety of test procedures and the results evaluated. The aggregates were of good, fair, or poor performance. A good aggregate was one that had a long prior history of satisfactory use in a particular application. A fair aggregate was one that had been used, but at the time of use or subsequently, there had been a suspicion or report of performance problems without a complete failure. A poor aggregate was one that had been used and resulted in an early failure of the pavement. Alternatively, a poor rating was given to an aggregate of such unsatisfactory composition (shale, very shaley limestone) that, if it had been used, failure would have certainly occurred. Senior and Rogers (5) describe this research in more detail.

Figure 3 shows Los Angeles abrasion and impact loss against Micro-Deval abrasion loss. It can be seen that the Micro-Deval test is good at separating the majority

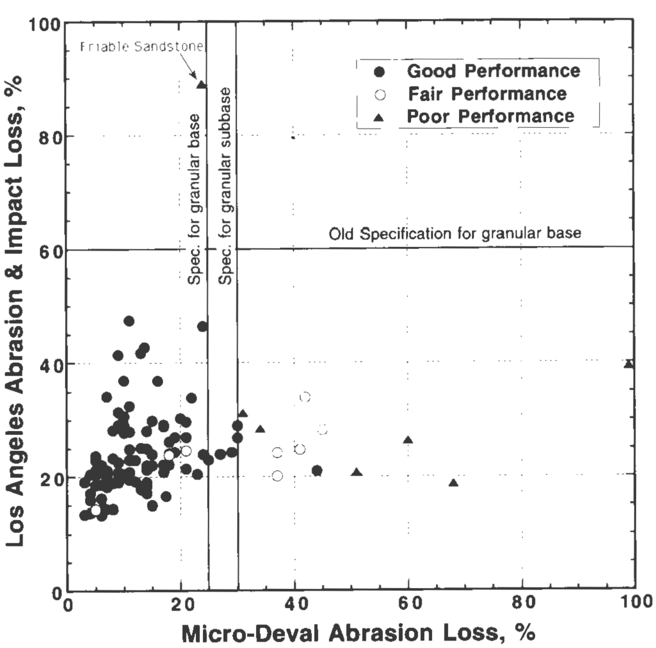


FIGURE 3 Los Angeles abrasion and impact loss value plotted against Micro-Deval abrasion loss value.

of poor aggregates from the good aggregates. Samples that are poor performers and fall at the boundary between the good and fair groups are shaley limestone and very shaley limestone. The one sample not detected by the Micro-Deval test was the friable sandstone from the Oliver Quarry. In 1994 MTO changed the specifications for granular base aggregates. The Los Angeles abrasion and impact test and petrographic examination requirement was abandoned, and a maximum Micro-Deval test loss of 25 percent for granular base and 30 percent for granular subbase was instituted. In addition, there are requirements that the material be nonplastic and that mica not constitute more than 10 percent of the material on the 75-μm sieve. Aggregates that contain more than 10 percent mica may be used if the field performance or results of permeability testing are satisfactory.

Conclusions

Further improvements of aggregate test methods that will provide more precise correlation with field performance and will be able to identify marginal or problem aggregates are being developed and implemented by the MTO to alleviate granular base failure problems. Placing good-quality aggregate materials in Ontario's low-volume roads is a good investment since many of these roads will become high-volume roads.

ACKNOWLEDGMENTS

The authors wish to acknowledge the observations and hard work of their colleagues in the highway construction and testing business, who are always looking for cases of unexplained failure and have recognized their importance to understanding the performance of materials. Without their assistance and help, this paper would not have been possible.

REFERENCES

1. Callaghan, N., J. d'Etremont, R. Lindsay, and N. Balfour. *Footpaths to Freeways.* Ministry of Transportation and Communications, Toronto, Ontario, Canada, 1984.
2. Roads and Transportation Association of Canada. *Canada's Roadway Infrastructure: Selected Facts and Figures.* Roads and Transportation Association of Canada, 1990.
3. Rogers, C. A. Frost Action as the Cause of Dolostone Popouts in Asphalt, Kingston, Ontario. *Canadian Geotechnical Journal,* Vol. 20, 1983, pp. 542–546.
4. Tourenq, C. L'essai Micro-Deval. *Bulletin Liaison Laboratoire Routières, Ponts et Chaussées, Paris, France,* No. 50, 1971, pp. 69–76.
5. Senior, S., and C. A. Rogers. Laboratory Tests for Predicting Coarse Aggregate Performance in Ontario. In *Transportation Research Record 1301,* TRB, National Research Council, Washington, D.C., 1990, pp. 97–106.

Recycling of Road Surfaces with the Roto Trimmer Mobile Rock Processor

Donna McConnaha Sheehy, *USDA Forest Service, Missoula, Montana*

How can we recycle existing road materials in place to reduce their size and develop a tough, long-lasting, maintainable cushion that will weather well and cost substantially less than importing materials processed elsewhere? Current methods of reconditioning worn-out native-surfaced roads have had limited success and can be quite costly. Development of aggregate pit sites for resurfacing roads can also be an expensive and lengthy process. In addition, there may be environmental impacts connected with opening the pit and any associated road construction or reconstruction. Since 1990 the Northern Region of the USDA Forest Service has been utilizing and evaluating a machine called the Roto Trimmer mobile rock processor. It has been used to recondition approximately 185 m (115 mi) of worn-out native-surfaced roads with varying degrees of success in Montana, Idaho, Wyoming, and Alaska. This process is expected to become a viable tool for reconditioning existing roads and provide future cost savings and environmental benefits.

The Northern Region of the USDA Forest Service—which includes Montana, portions of North and South Dakota, and northern Idaho—contains more than 78 100 km (48,500 mi) of roads of which 82 percent is native-surfaced, 16 percent is aggregate surfaced, and 2 percent is paved. With years of historically low maintenance budgets, many of these native-surfaced roads are worn out and not maintainable. The existing roadbeds are extremely rough and composed of cobbles, ledge rock, and boulders, with little or no fines. There is a great need for a maintainable road.

Traditionally, roads of this type were maintained either by ripping, grading, and grid rolling or by importing aggregate surfacing. Ripping and grading do not always result in a satisfactory end product. Resurfacing requires either hauling material from existing pits or developing new pits. Development of new material sources requires the proper permits, royalty fees, and environmental assessments or impact statements and can become quite expensive and involve possible delays. In addition, pit sites can have undesirable impacts on the environment.

In 1990 the Northern Region began experimenting with an alternative method of reconditioning worn-out, native-surfaced roads. The Roto Trimmer mobile rock processor was first used to recondition 37 km (23 mi) of road on the Lolo National Forest. Since then the machine has been used on several projects throughout the region and in Alaska. The Canadian government has also expressed an interest in the machine and the process. The region is continuing to utilize this process to produce a maintainable road as it has become a viable, cost-effective tool for road maintenance.

Equipment

The process of recycling existing road surfaces involves the use of the Roto Trimmer mobile rock processor, usually followed by the use of a motor grader of 135 hp or greater, an Elliot grid roller, a traditional vibratory roller, and an optional water truck.

The original model for the Roto Trimmer was a rock crusher developed by Crude Tool Works of Kenai, Alaska. The 1991 purchase price was $196,500. This machine was designed to break up permafrost and had not been utilized as a road maintenance device.

Triple Tree Inc., of Missoula, Montana, purchased the rock crusher and made more than $45,000 worth of modifications to the machine in order to process road surfaces more effectively. A patent on these modifications is pending.

The machine, shown in Figure 1, resembles a giant Rototiller and consists of a two-component kit: a front rotary drum attachment and a rear power pack. This kit mounts on any suitable carrier such as a loader, grader, or scraper. It is essentially balanced as the drum and the power pack each weigh approximately 5 450 kg (12,000 lb). The kit components can be mounted or removed from a loader in approximately 12 person-hr.

Rotary Drum Attachment

The rotary drum attachment consists of the drum and a 76-mm (3-in.), removable, solid steel rear-impact plate. The rotary drum is 0.9 m (3 ft) in diameter by 3 m (10 ft) wide and can turn in forward and reverse motion at 84 rpm. It can also be raised for slope work. Three types of drums have been constructed: one that produces a level surface, one that produces a windrow to the middle, and one that produces a windrow to the sides.

The drum has 184 carbide-tipped teeth in knuckle holders mounted in a spiraled-inward pattern, as shown

FIGURE 1 Roto Trimmer mounted on a Caterpillar loader (rotary drum on front lift arms and power pack at rear).

FIGURE 2 Rotary drum attachment with carbide-tipped teeth.

in Figure 2. The teeth rotate after every strike to produce even wear. Teeth have an average life of 8 hr and are the only routine replacement item. They are quickly removed with either a forked tool and hammer or an air drill and are easily installed with a tap of the hammer. Teeth cost approximately $3.20 each in 1993.

A tooth hits every 3.175 mm (0.125 in.) of soil. The teeth crush the rock and rip the material as the drum rotates. The loose rock revolves counterclockwise to the impact plate on which it is further fractured and blended.

Power Pack Attachment

The power pack attachment includes a Caterpillar Model 3406 (D-9) diesel engine rated at 400 hp, a hydraulic pump with reservoir and related hardware, and work lights and taillights. The power pack allows the machine to operate at temperatures from −45°C to 26°C (−50°F to 80°F).

Machine Modifications

After the first project, the Roto Trimmer was modified by the road contractor, Triple Tree Inc., to increase its efficiency and to produce a better end product that met gradation specifications (*1*). These modifications resulted in better rock fragmentation and decreased the outcasting of larger rocks. They also added further protection and stability, simplified the machine's operation, and increased its maneuverability.

Machine Capabilities

The machine processes most existing road surface materials to a specified depth of 0 to 150 mm (0 to 6 in.)

and a gradation of −100 mm (4 in.). It cuts a path 3 m (10 ft) wide and leaves the road material in a well-mixed state.

Technical Specifications

Work is controlled by a special project specification developed for use in the Northern Region. This specification describes the process and the desired end product. It is modified as needed. The specification and an equipment specification developed for some contracts to provide additional quality assurance are shown in Figures 3 and 4.

Measurement for contract payment has usually been by slope distance along the centerline of the road. Payment is either by kilometer, by station, or by lump sum. Some contracts were simply for equipment rental by the hour.

Operation

Generally no preparatory work is required other than blasting or breaking of large oversized rock or outcroppings to save on tooth breakage. A handheld Pionjar rock hammer is used where blasting is not an option.

The optimal procedure has been to operate the machine at idling speed against the grain of the rock. A pass 3 m (10 ft) wide by 100 to 150 mm (4 to 6 in.) deep is made (see Figure 5), followed by another pass alongside for most roads wider than 3 m (10 ft). When attempting to grind to a depth of 150 mm (6 in.), the machine will usually make two passes to obtain full breakage of the rock material. Small sidecast windrows are made, which are reripped to produce material of a smaller particle size. A grader smoothes the windrows, a water truck applies compaction water and controls the minimal dust, and a traditional vibratory roller used in conjunction with an Elliot grid vibratory compactor brings up fines and compacts the soil. The result is a finished road with a hard, smooth, maintainable driving surface.

This process works well in frozen soil, as the rocks stay in place for crushing. Some dampness in the soil is preferred for decreased machine wear, dust control, and compaction.

Material Quality

The quality of the recycled road material cannot be regulated to the extent that the quality of crushed or screened material can. However, tests have shown that there is a substantial improvement to most materials processed by this recycling method. The materials are reduced to approximately the 100-mm (4-in.) class with a large increase in fines. This, combined with adjustable depth control and good compaction, provides an excellent, long-lasting, and maintainable road cushion material (see Figure 6). It appears that roads would not need to be recrushed for 6 to 10 years.

Project Areas

Since 1990, approximately 185 km (115 mi) of roads have been reconditioned with this process with varying degrees of success. A sampling of these projects follows.

Gilbert/Schwartz Creek Demonstration Project

The Gilbert/Schwartz Creek project was located in Lolo National Forest, Missoula, Montana, in 1990.

Project Description

This demonstration project consisted of reconditioning portions of 37 km (23 mi) of very rough, worn-out roads with excessive rock and little or no fine material. The road was built in 1963 and had little use and no maintenance since that time. Sample sites were chosen in a variety of rock types common to the area, including massive limestone/dolomite outcrops and boulders with some to no fracturing; fractured, brittle metaquartzite; and an igneous intrusion. Samples were taken before and after processing and tested for plastic and liquid limits, plasticity index, abrasion, sieve analysis, and durability (*1*).

Project Results

The test results showed significant improvements in gradation and a substantial reduction in rock greater than 75 mm (3 in.). The oversized rock was fragmented, and an appreciable amount of fines was produced. There was a thorough mixing of the processed material, which tends to reduce pothole development. A few isolated sections had noticeably oversized material. The sections showing the greatest improvement had rocks that displayed weathering, internal fracturing, or jointing (*1*).

Project Costs

Time and cost comparisons were difficult to make since this was an experimental project. Operating costs and production rates were reasonably close to those of conventional methods. Machine operating costs were approximately $52 per 300 m (100 ft) of road for the

Special Project Specification

Section 815 - Reestablishing Native Surface

DESCRIPTION

815.01 Work This work shall involve using state-of-the-art mobile, rock milling equipment to manufacture cushion material to a specified depth on a native surface road. The surface shall be well blended. Work shall include reducing bedrock and oversized rock down to a specified size. All oversize material shall be broken down and incorporated into the road, or sidecast to the downhill side of the road. It also includes watering, mixing, rolling, blading and shaping of the manufactured cushion material into a smooth, non-segregated road surface.

CONSTRUCTION

815.02 Method The cushion material shall be produced by processing the existing surface as **SHOWN ON THE DRAWINGS**. The cushion material shall be developed by blending, breaking, grinding and pulverizing bedrock and oversized rock contained within those dimensions down to the maximum size of 100 mm (4 in) or as specified in the drawings.

The cushion material shall have the fine and coarse particles uniformly mixed across the entire width of the road to eliminate localized pockets of open graded rock or fractured ledge rock.

The floor beneath the native surface cushion shall be uniform across the total width of the road such that the required

FIGURE 3 Special Project Specification for reestablishing native-surfaced roads.

depth of 150 mm (6 in) (or as specified) is reasonably uniform to meet the road typical section. No bedrock or oversized rock will be allowed within the specified depth of the cushion material. Oversized material shall be disposed of as designated by the engineer.

In limited situations, when approved in writing by the Engineer, the contractor may elect to cover segments of bedrock with imported material. This imported material must meet all the requirements of the specified cushion material and its source must be approved by the Engineer. There shall be a minimum grade transition of 15 m (50 ft) on each side of the covered section. Approval of this option will be infrequent and will not include segments of loose rock or boulders.

815.03 Compaction After development of the cushion material, the traveled way and shoulders shall be uniformly crowned, insloped, outsloped, or superelevated as **SHOWN ON THE DRAWINGS**. The entire roadway shall be compacted by operating compaction equipment over the full width of the layer until visible deformation ceases. At least three complete passes shall be made. Compaction equipment shall be capable of obtaining compaction requirements without detrimentally affecting the compacted material.

815.04 Water While compacting, blending and finishing, the cushion material shall have sufficient moisture, either natural or added, to prevent segregation and to facilitate compaction.

815.05 Finishing At the conclusion of the blading, shaping, and compaction operations, the roadbed shall have a uniform grade and smooth surface free of hollows, depressions, segregation and projections above the road surface. No berm or ridge shall be left along either shoulder of the roadbed, unless directed by the engineer.

FIGURE 3 (*continued*)

(a) - Road Reclaimer and Carrier

(1) Carrier must be capable of carrying and maneuvering cutting head at required depths.

(2) Cutting head weight shall be 6,800 kg (15,000 lb) or more to avoid excessive bouncing.

(3) Rotary drum shall have at least 184 carbide teeth set in a spiral inward pattern.

(4) Rotary drum should be able to make a 3 m (10 ft) wide cut and rotate at 94 rpm's minimum and exert a force of 34,450 kPa (5,000 psi).

(5) Power plant shall be a minimum 400 hp, have hydraulic pump with reservoir and related hardware.

(6) Manufacture date shall be 1990 model or newer.

(b) - Road Grader

(1) Grader shall have a 3.7 to 4.3 m (12 to 14 ft) hydraulic moldboard equipped with serrated blade and appropriate end pieces to sift out oversized material.

(2) Basic operating weight shall be 13,170 kg (29,000 lb) or more.

(3) Have a ripper attachment.

(4) Minimum rated flywheel horsepower shall be 135 hp.

(5) Manufacture date shall be 1977 or newer.

FIGURE 4 Equipment specification for use with Special Project Specification.

(c) - Water Truck

(1) Minimum capacity of 11,355 L (3,000 gal). .

(2) Shall have a spreader bar. Water application may be gravity fed or pressurized.

(3) Draw or suction hose for filling must reach a minimum of 10.7 m (35 ft).

(4) Manufacture date shall be 1977 or newer.

(d) - Vibratory Compactor

(1) Compactor shall have minimum 22,700 kg (50,000 lb) dynamic force.

(2) Dual amplitude and variable frequency of 0-265 Hz.

(3) Compactor shall be vibrating smooth steel drum with Elliot Grid or comparable system.

(4) Date of manufacture 1985 or newer.

(e) - D6 or D7G Dozer

(1) Must have ripper and dozer.

(2) Must have at least 170 to 200 hp.

(3) Manufacture date 1977 or newer.

FIGURE 4 *(continued)*

FIGURE 5 Roto Trimmer making first pass.

Roto Trimmer, a grader, a vibratory roller, and a water truck. The contract for this project cost about $2,500/km ($4,000/mi).

Eagle Creek Road Reconditioning Project

The Eagle Creek project was performed in 1991 in the Gallatin National Forest, Bozeman, Montana.

Project Description

This project consisted of reconditioning 9 km (5.6 mi) of road on which the surfacing had worn away, leaving very hard, protruding rocks ranging in size from 150 to 400 mm (6 to 16 in.). Little or no fines were present.

Project Results

Many of the protruding rocks were broken during the rototilling process. Others were pulled up by the grader and sidecast. Although a 4-in. minus surface was not obtained in all cases, sufficient fines were produced to allow the grader to blade and restore the road to its original template. In addition, the ditches were pulled and the turnouts were milled, bladed, and rolled. The finished road surface averaged 4.6 m (15 ft) in width. The finished product has not required any additional maintenance blading for the last 2 years.

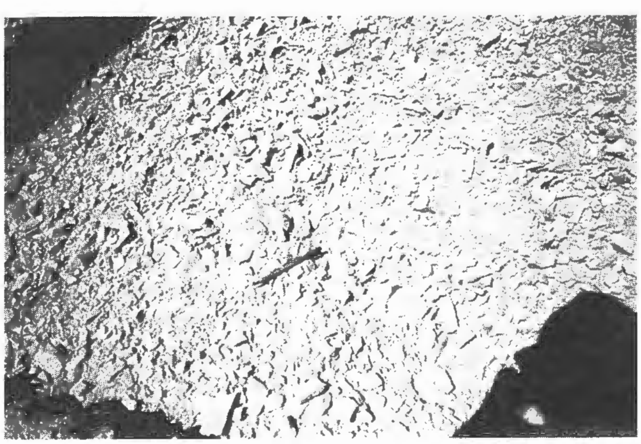


FIGURE 6 Typical road surfacing after being processed with Roto Trimmer.

Project Costs

Average production rates were 0.19 km/hr (0.12 mi/hr) for the Roto Trimmer, 0.18 km/hr (0.11 mi/hr) for the grader, and 0.18 km/hr (0.11 mi/hr) for smoothing with the vibratory roller. Average costs were approximately $2,730/km ($4,400/mi). The total cost was $24,987.50, based on a time-and-equipment contract.

For the sake of comparison, an estimate was made for the cost of placing pit-run material on the project. Placing 5,200 m^3 (6,800 y^3) of material hauled from a distance of 5 km (3 mi) and adding a 10 percent mobilization factor would have resulted in a total cost of $58,800.

Comments

Many oversized rocks were sidecast by the Roto Trimmer. The contractor indicated that on the basis of the size and hardness of the material, the oversized material should have been ripped with a Caterpillar bulldozer and hoe before being processed with the Roto Trimmer.

Forestwide Road Reconditioning Project

The Idaho Panhandle National Forest, Coeur d'Alene, Idaho, was the location of a reconditioning project in 1993.

Project Description

The project consisted of reconditioning approximately 39 km (24 mi) of native-surfaced roads and 3.2 km (2 mi) of a chipsealed road. Roads were located throughout the forest. Road widths varied from 3.7 to 5.5 m (12 to 18 ft) with turnouts. The roads exhibited rough surfaces, large rock protrusions, and exposed bedrock, and most fines were worn off.

The roughness (rideability) of the roads was measured with a Cox roughness meter before and after processing by the Roto Trimmer. In addition, gradations were obtained before and after processing (2).

Project Results

Overall, the recycling process proved very successful. Instead of blasting, a Pionjar rock hammer was used to fracture some of the more difficult rock. This allowed the Roto Trimmer to move through the rock sections more easily, resulting in cost savings. The native-surfaced roads were worked to a depth of 150 mm (6 in.) and the chipseal surface was worked to a depth of between 100 and 150 mm (4 and 6 in.). The maximum-size rock allowed in the cushion material was 100 mm (4 in.). Most of the project roads are adjacent to streams or rivers. As the Roto Trimmer did not throw rock or generate excess dust, there was no noticeable impact on drainage during reconstruction (Figure 7).

The roughness counts were reduced by 50 to 70 percent. This reduction is equivalent to changing from a rough or an extremely rough surface to a moderately rough surface. Before processing, the speed of the roads was controlled by the rough character of the surface. After processing, the speed is controlled primarily by the alignment (2).

In general, the existing gradation of 250 to 300 mm (10 to 12 in.) minus was reduced to 75 to 100 mm (3 to 4 in.) minus. The roads can now be graded where previously it was not possible (2).

Project Costs

Average production rates were 1.2 km/day (0.75 mi/day). Average production costs were approximately \$3,650/km (\$5,900/mi), not including mobilization and costs for moving between work sites. Meeting the specifications and the equipment requirements resulted in a total project cost of \$169,300.

Comments

Working the chipseal surface was an experiment to determine what type of finished product would be generated. Under one section of chipseal was a gravel surface. The end result was a good, finished surface with a well-mixed, chipseal and gravel material. Another section of the chipseal was mixed with a latex material. This surface caused some problems and was difficult to process.

FIGURE 7 Roto Trimmer in operation; note small amount of dust and sidecast material.

Some of the roads within this project required spot work on rocky sections only. It appears that doing work in spots rather than in a continuous length was more expensive and harder to accomplish.

Round rock caused some problems in that the drum could not break it down. The rock would roll around in the drum until it was sidecast.

Potential effects on water quality were reduced as the higher-standard surface resists traffic-induced damage and erosion. Other environmental benefits and significant cost savings were realized as no pit development and hauling of surfacing material were required.

Leigh Lake Road No. 4786

The Leigh Lake Road project, located in the Kootenai National Forest, Libby, Montana, was performed in 1993.

Project Description

The project consisted of reconstruction of 2.7 km (1.7 mi) of a road that had not been maintained but received a large amount of recreational use, primarily by high-clearance vehicles. Over the years, the road had become a trench with no drain dips or open tops and an occasional nonfunctioning culvert. The surface was quite rough—many of the users would walk rather than drive on the road. There were several concerns over the reconstruction: how to maintain the integrity of the road with minimum clearing; what to do with the outside shoulder that was primarily a disposal area for rocks; how to provide adequate drainage; and where to obtain a suitable source for surfacing the road. Figures 8 and 9 show conditions before and after processing of the road surface with the Roto Trimmer.

Project Results

The project was considered quite successful. The native-surfaced rock fractured to a fairly uniform 100 mm (4 in.) pit-run surface, eliminating the need for a borrow source. The material in the outside berm was utilized in place, so disposal of oversize material was kept to a minimum. The reconstructed road maintained the integrity of the original location but with a drivable, maintainable surface. The road was shaped to drain with open tops.

FIGURE 8 Leigh Lake Road: typical section before processing with Roto Trimmer.

FIGURE 9 Leigh Lake Road: typical section after processing with Roto Trimmer.

Project Costs

The total cost for the project was $32,655: $1,800 for clearing; $11,355 for mobilization; and $19,500 for reestablishing the native surface. Mobilization included an excavator, Roto Trimmer, a grader, and a roller.

Comments

Mobilization costs would have been much less if several projects had been located within the same area. This single project took place approximately 320 km (200 mi) from the construction company's location.

SUMMARY

Limitations

There are limitations to the materials that can be processed. Most large metamorphic or igneous boulders are not easily reduced and may require some preprocessing. Some are just too hard. Generally, if the material can be ripped with a D8 size bulldozer, then it can be processed by the Roto Trimmer.

Round rocks cause some problems in that they become stuck between the teeth or roll around in the drum until they are sidecast, rather than being processed.

Benefits

Economics

In most cases the cost of Roto Trimming is a fraction of the cost of crushing material or hauling pit-run material to the site. Operating costs and production rates appear to be reasonably close to those of the conventional methods of ripping, grading, and grid rolling.

This process eliminates the road surface memory, which may contain soft spots, potholing, and washboarding. The cushion and subsurface have been trimmed to a smooth flat plane that will dissipate most moisture laterally and eliminate previous problem areas. The road surfaces produced with this recycling process should make for less maintenance and a longer life, resulting in long-term cost savings.

Environmental Impacts

The recycling of road surfaces eliminates the need for opening new material sources and any associated new road construction. As shown in Figure 7, the actual reconstruction process generates less dust and sidecasting than traditional methods, resulting in fewer impacts on adjacent drainages and any associated fisheries. It utilizes the existing materials in the road surface to build a stable cushion. When the material is compacted with a vibratory roller, a finished product is developed that will last for years without harming the environment.

Other Applications

Aggregate-Surfaced Roads

The process of reconditioning native-surfaced roads should also apply to crushed or pit-run road surfaces that may be experiencing rutting, washboarding, potholing, or material segregation. Binders could be added that would lengthen the life of the road.

Road Stabilization

When roads are no longer needed for resource management, they need to be closed or obliterated and the existing surfaces stabilized to prevent erosion. Scarifying an inch of the native surface material produces a roadbed ready to be seeded and fertilized. Because the process leaves the road surface in a well-defined and shaped cross section, the seeded roadbed can still provide access during emergencies such as forest fires.

ACKNOWLEDGMENTS

The author would like to acknowledge the cooperation of the following individuals in supplying the relevant data and information: Mary Northrup and Jim Northrup of the Idaho Panhandle National Forest, Paul Hayes of the Gallatin National Forest, Darrel Steinberg of the Lolo National Forest, Donna Tucker of the Kootenai National Forest, and Bob Greil, Dick Greil, Tom Greil, and Jim Greil of Triple Tree Inc., Missoula, Montana.

REFERENCES

1. Hegman, S., and K. Kreyns. *Field Testing of Roto Trimmer Mobile Rock Processor.* USDA Forest Service Engineering Field Notes, Volume 23, September–October 1991.
2. Northrup, J. *Development of Cushion Aggregate on Native Surfaced Roads by Use of the Roto Trimmer.* USDA Forest Service, 1994.

WASTE MATERIALS

Tire Chips in the Base Course of a Local Road

Ronald I. Frascoia and Robert F. Cauley, *Vermont Agency of Transportation*

A yearly occurrence in the state of Vermont is affectionately termed the "mud season." Although this phenomenon is often viewed in a joking manner, the condition of the rural gravel roadways in Vermont every spring creates considerable distress and many times hazards for the rural traveling public. This paper presents a discussion of the installation of a layer of tire chips beneath the gravel surface of the roadway. It has been found that the tire chips interrupt ground water capillary action and provide for drainage of water seeping through the roadway surface. The resulting roadway has been found to be substantially firmer and more passable. Even though vehicle loads must be held to a minimum, the overall objective of reducing distress and hazards caused by muddy rural roads appears to have been accomplished.

Pressures are mounting on the transportation community to develop procedures that will accomodate the use of waste scrap tires in highway applications. Mandates include specific requirements for tire rubber use in bituminous mix or asphalt chip seals and stress-absorbing interlayers. The disadvantages of utilizing rubber in asphalt include the high cost of such treatments and the low rubber consumption rates. Other more practical and less costly ways are needed to effectively use up the waste tire stockpiles and the additional tires being added to the waste stream annually.

This paper describes a promising alternative use that can consume a high volume of tires with the potential for cost savings for the user.

Treatment

The alternative use of tire chips as a base-course layer began in 1990 in the town of Georgia, Vermont, with the construction of a 100-m (330-ft) test section on Town Highway (TH) No. 4. The shredded tires were designed to serve as both a drainage layer and a barrier to prevent contamination between a wet silty sand subgrade and the gravel base. The initial success of the treatment, described in Vermont Report U91-06, led to the construction of additional segments in the following 3 years.

Location and Materials

The tire chip base was constructed on TH 4, known as the Oakland Station Road, in the town of Georgia, Vermont. This Class II highway begins where it intersects Route 7 at mm 2.50, 1.6 km (1 mi) north of Exit 18 on Interstate 89 and continues north 5.4 km (3.36 mi) where it terminates at its intersection with TH 1, approximately 0.6 km (0.35 mi) west of Vermont Route 104. The test section starts approximately 2.1 km (1.3 mi) north of the Route 7 intersection and extends north for approximately 760 m (2,500 ft).

The original roadway consisted of approximately 60 cm (2 ft) of gravel on a silty sand subgrade. Gradation tests on samples of the subgrade material revealed from 24 to 43 percent passing the 0.075-mm (No. 200) sieve.

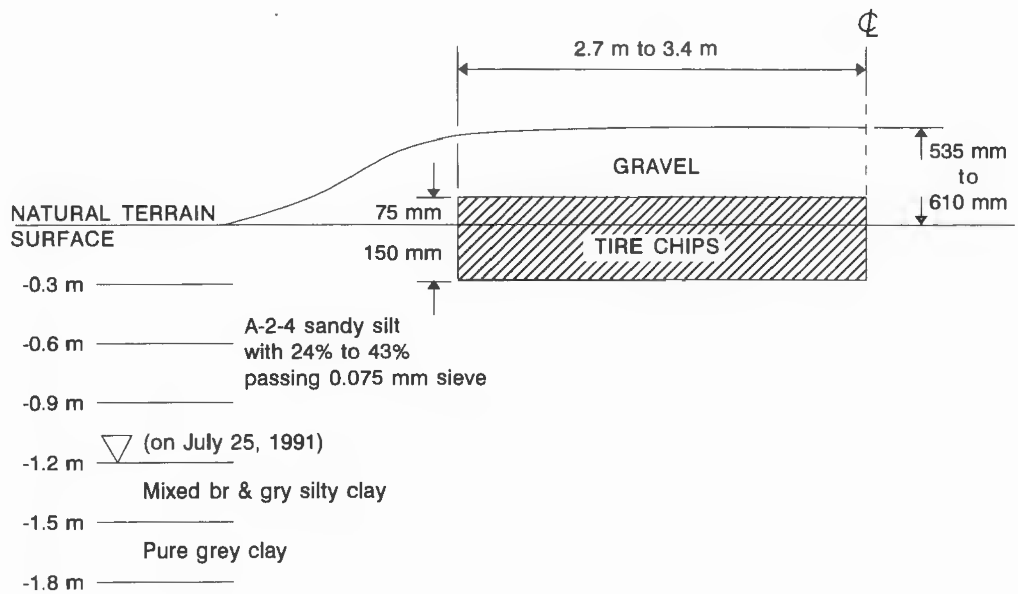


FIGURE 1 Typical section of TH 4, Georgia, Vermont: initial construction.

A high water table commonly resulted in the area's becoming impassable for two-wheel-drive vehicles during the spring "mud season." The pumping action of the traffic had resulted in contamination of the gravel with fine materials. The traffic is in the range of several hundred vehicles per day, with much of it consisting of early-morning and late-afternoon commuters, plus some heavy vehicles such as milk trucks.

Construction Procedure

The construction process included removal of the existing gravel with a backhoe, removal and disposal of approximately 150 mm (6 in.) of silty subgrade material, and backfilling with tire chips. The chips were placed with dump trucks and leveled in a 230- to 305-mm (9- to 12-in.) course with the backhoe. Replacement of most of the original gravel and the addition of several centimeters of new gravel completed the process.

The initial construction in 1990 (Figure 1) included 38 m^3 (50 yd^3) of large tire shreds, of a nominal size of 100 mm × 200 mm (4 in. × 4 in.), which had passed through the chipper once, and 115 m^3 (150 yd^3) of small shreds, of a nominal size of 50 mm × 50 mm (2 in. × 2 in.), which had been chipped two or three times. The chips were transported by town trucks and stock-

FIGURE 2 Dumping tire chips, August 1, 1990.

FIGURE 3 Leveling tire chips with backhoe.

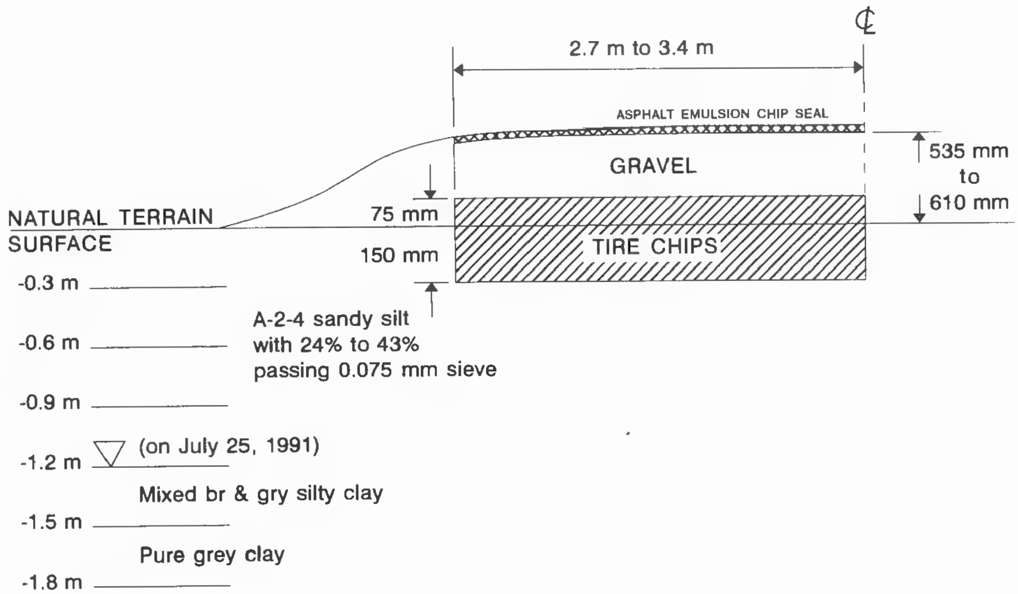


FIGURE 4 Typical section of TH 4, Georgia, Vermont: fall 1992.

piled near the construction site. In subsequent years, only 50-mm (2-in.) chips were utilized and the material was hauled directly from the chipping site in 9.2-m^3 (12-yd^3) dump trucks with sideboards that increased the capacity to 12.2 m^3 (16 yd^3).

In the autumn of 1992, a chip seal (Figure 4) was placed over the initial section of roadway treated in 1990. The process included the placement of a layer of pea stone, a 4.5-L/m^2 (1-gal/yd^2) application of asphalt emulsion, and a mixed 6.4-mm (1/4-in.) and 9.5-mm (3/8-in.) stone surface topping.

Cost Information

The tire chips were purchased from Palmer Shredding, Inc., North Ferrisburg, Vermont, at a cost of \$1.30/m^3 (\$1.00/yd^3). Purchases included 150 m^3 (200 yd^3) in 1990, 220 m^3 (288 yd^3) in 1991, 325 m^3 (426 yd^3) in 1992, and 440 m^3 (576 yd^3) in 1993. Replacement gravel was purchased at a cost of \$5.00/m^3 (\$3.85/yd^3).

Testing and Observations

An inspection of the initial test section on August 30, 1990, revealed the existence of some fine longitudinal cracks in the surface of the gravel roadway. The cracks were noted at eight locations, totaling approximately 19 m (63 ft). A few short transverse cracks were also noted extending off the longer longitudinal cracks. There was no detectable rutting in the wheel path areas.

The test section and adjacent roadway were free of any additional distress when observed on November 20, 1990. At that time, slotted polyvinyl chloride (PVC) well-monitoring pipe 50 mm (2 in.) in diameter was installed at two locations along the easterly toe of the roadway. The water table was found to average 440 mm (17.5 in.) below the ground surface or 290 mm (11.5 in.) below the bottom of the tire chips.

On April 3, 1991 (traditionally, about the start of the mud season), the roadway was inspected, photographs were taken (Figure 5), and the water table ele-

FIGURE 5 Overview of tire chip section, April 3, 1991. Note surface moisture on adjacent untreated sections.

vation was measured. The roadway surface within the test section was visibly dry and free of any rutting. A few fine, longitudinal and transverse cracks were visible on the northerly half of the test section, with most noted in the southbound lane. The water table averaged 305 mm (12 in.) below the natural terrain surface or 151 mm (six in.) below the estimated bottom of the tire chips. The water table elevation was 140 mm (5.5 in.) higher than that recorded the previous November.

By comparison, the untreated roadway portions north and south of the tire chip section were in poor condition. The surfaces were visibly wet and revealed numerous ruts, cracks, and boils. The poor areas were soft to walk on and water could be drawn to the surface with a tamping action. It must be noted, however, that in general, TH 4 was in better overall shape than in other years, because of a milder winter with below-average snowfall and a dry spring season.

On July 25, 1991, construction of another segment was observed and photographs were taken (Figures 6–9). At that time, the backhoe was utilized to place two additional monitoring pipes and to dig a test pit in the year-old tire chip section. The soil at the well sites began as a sandy silt that approached saturation approximately 120 cm (4 ft) below the road surface elevation, changed to a mixed brown and grey silty clay at a depth of 150 cm (5 ft), and became a pure grey clay at a depth of 180 cm (6 ft) (Figures 1 and 2).

The 60-cm × 180-cm (2-ft × 6-ft) test pit was dug in the left wheelpath of the southbound lane, 7.6 m (25 ft) north of the southerly well point placed in 1990 (Figures 10 and 11). The pit revealed 530 mm (21 in.) of gravel over 200 to 230 mm (8 to 9 in.) of tightly compacted tire chips. The top 150 mm (6 in.) of gravel contained 8 percent minus 0.075 mm (No. 200) sieve material, whereas the remainder averaged 15 percent minus 0.075 mm material. The top side of the chips

FIGURE 7 Backfilling excavation with 16-yd^3 load of tire chips.

FIGURE 8 Leveling tire chips with backhoe to obtain ±8-in. layer.

FIGURE 6 Roadway excavation included stockpiling of ±22 in. of existing gravel and disposal of 8 to 12 in. of silty sand subgrade.

FIGURE 9 Backfilling trench with original gravel.

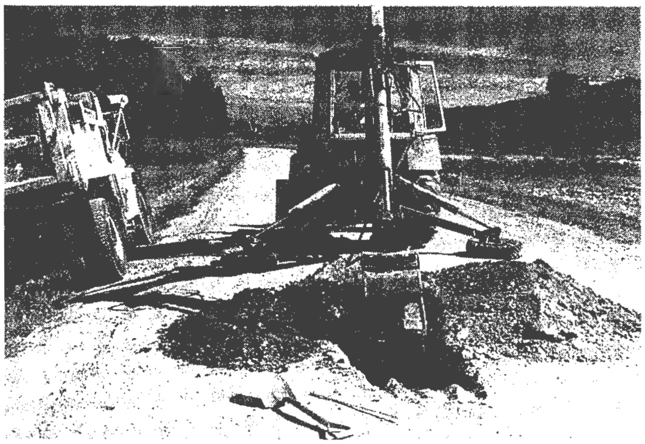


FIGURE 10 Excavating test pit in 1990 construction project.

showed a trace of moisture whereas the bottom side was dry. The silty sand beneath the chips contained enough moisture for it to hold together when squeezed (Figure 12). No attempt was made to locate the water table elevation.

Falling weight deflectometer (FWD) tests were taken on the 2-month-old chip seal surface in November 1992. The test values indicated a significant deflection of the test surface, which would suggest that the tire chip layer allows too much deflection under heavy loads for any future bituminous pavement to survive without additional gravel cover over the tire chip layer.

On August 12, 1993, construction of another tire chip base segment was observed and the 11-month old chip seal was inspected. A nearly full-width transverse crack and a few random longitudinal cracks were visible in the northbound lane. A nearly continuous longitudinal crack extended for one-half the length of the treatment in the southbound lane. All of the cracks were very fine, and stone loss was minimal. Several bituminous patches had been placed along the centerline where there had been some loss of the seal at the construction joint.

On October 31, 1994, FWD readings were taken at four locations, two on gravel sections without tire chips (Test Sites 1 and 2) that now have a 75-mm (3-in.) bituminous concrete surface and two on the sections with tire chips (Test Sites 3 and 4) that now have a chip seal. Deflections and structural numbers were as follows:

Test Site	*Deflection (mm)*	*Structural No.*
1	0.73	2.80
2	0.87	2.52
3	2.60	1.61
4	3.27	1.49

These data should be considered primarily in a qualitative manner when the test sections are compared. Calibration of the FWD is valid only up to deflections of 2 mm (80 mils). The measurements confirm that vehicle loads on the tire chip sections must be confined to the traditional (existing) automobile traffic with an occasional milk truck.

Summary and Conclusions

The construction of a test section of highway with a tire chip base in the town of Georgia, Vermont, and its performance to date can be summarized as follows:

- Approximately 65,000 tires chipped to a 50-mm (2-in.) size were placed on 760 m (2,500 ft) of TH 4 in Georgia, Vermont.

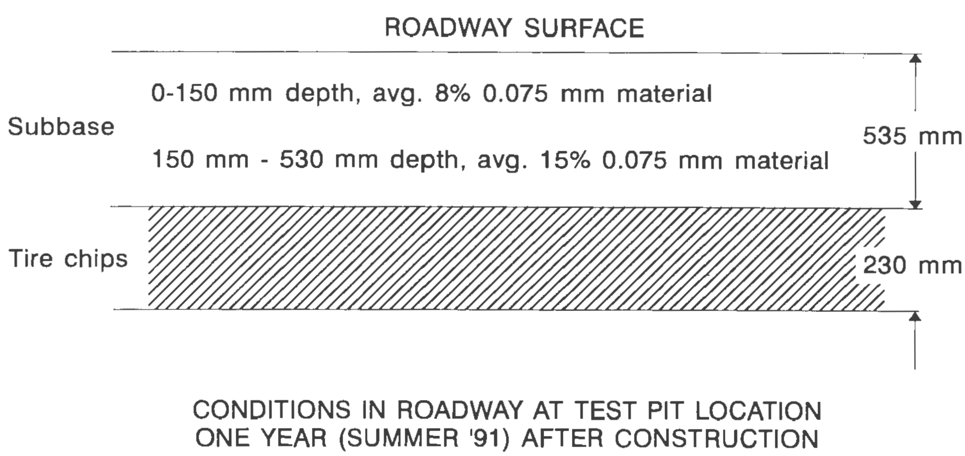


FIGURE 11 Conditions on roadway at test pit location one year after construction (summer 1991).

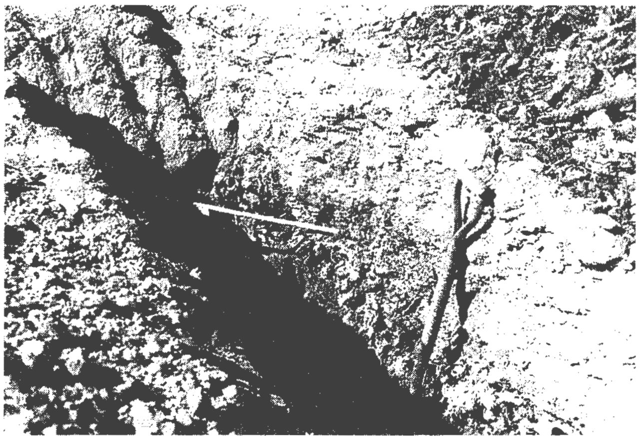


FIGURE 12 Base consisted of 20 to 22 in. of dirty gravel over ±8 in. of compacted tire chips; gravel was dry, whereas silty sand subgrade contained enough moisture to hold together when squeezed.

- The tire chips provided a tightly compacted layer averaging 230 mm (9 in.) thick beneath 530 mm (21 in.) of poor-quality gravel that contained an average of 15 percent minus .075 mm (No. 200) sieve material.
- The tire chip layer enhanced the poor-quality gravel by cutting off the capillary rise of subsurface water and by reducing the moisture content of the gravel through good drainage.
- The muddy road conditions prevalent in past spring seasons did not recur following the placement of the tire chip layer.
- The use of tire chips at a cost of $1.30/m^3 ($1.00/yd^2) reduced the need for additional gravel that would cost $5.00/m^3 ($3.85/yd^3).
- An asphalt emulsion chip seal placed on the initial test section revealed only minor distress through its first year of service.
- FWD test values suggest the tire chip layer will allow too much deflection under heavy wheel loads for a bituminous pavement to resist cracking, unless an additional layer of gravel is placed over the tire chips.

Recommendations

The Agency of Transportation should encourage cities and towns to utilize tire chip layers in town highway bases, with an emphasis placed on areas where moisture conditions are a problem and future paving is unlikely. If the cost of hauling tire chips makes some potential locations unsuitable, consideration should be given to the concept of collecting tires at regional solid waste sites and shredding them with portable equipment.

Additional deflection testing is warranted, with emphasis placed on the comparison of values for similar gravel segments with and without the tire chip layer.

Followup

Monitoring will continue on the tire chip base and additional reports will be prepared when significant information is obtained.

Use of Woodwaste for Road Construction in Southeast Alaska

Cole F. Mullis, *Oregon Department of Transportation*
John K. Bowman, *USDA Forest Service, Petersburg, Alaska*

Wood fibers have a long history of use in road construction across the United States and British Columbia. They have been used in highway and low-volume road construction to reduce landslide potential and to cross settlement-sensitive areas. The USDA Forest Service has used wood fibers in many forms as an embankment material and as an alternative surfacing material. Many forms of wood fibers may be used in construction, including brush, construction slash, chunkwood, and sawmill residue such as bark, sawdust, and planer shavings. The history of wood fiber use is discussed and a recent USDA Forest Service demonstration project that used sawmill-generated woodwaste to construct 4 km (2.8 mi) of forest access road in Wrangell, Alaska, is outlined. The project was implemented to study the suitability of these materials for use in southeast Alaska. An evaluation of the engineering performance characteristics was conducted in an effort to determine some guidelines for future use. This evaluation focused on the rutting potential and road stiffness. A series of field and laboratory tests was conducted to address these issues. The main findings of the study are that the wood fibers perform adequately as both a driving surface and as a base layer for aggregate surfacing materials such as crushed or shot rock. When wood fibers are used as a driving surface, routine maintenance must be done to correct rutting and low-frequency washboarding. Blading may be done easily with a standard motor grader or a bulldozer.

Wood fibers have been used for a long time as a road building material in southeast Alaska. The use of logs and brush as a cover over organic soils and muskegs increased with the use of hydraulic-operated backhoe excavators in the late 1970s and early 1980s (*1*). Most logging roads constructed for the USDA Forest Service or other logging operators are constructed using a wood "debris mat." Most forest roads in southeast Alaska are first excavated to "pioneer grade," which is 60 cm (2 ft) below the designed subgrade elevation. This excavation usually occurs on side slopes and through cuts and can extend from 0 to 4 m (0 to 12 ft) deep at the road centerline. The excavated soil and rock are used to construct a bench, usually 7 to 8 m (20 to 25 ft) wide, upon which nonmerchantable trees, brush, limbs, and other construction slash are placed to form the debris mat. Blasted quarry rock is then placed by end dumping and spreading with a bulldozer to the designated subgrade elevation. A vibratory grid roller and motor grader may be used to finish the road surface or the road may be surfaced with crushed aggregate.

A number of experimental projects have used sawmill-generated woodwaste, including sawdust, bark, and shavings as a lightweight fill material. In the early 1970s, the state of Washington constructed a sawdust fill across a landslide area (*2*). In the 1960s and 1970s, British Columbia officials used wood fibers in three projects to cross settlement-sensitive areas such as peat and sensitive clay soils (*3*). In the 1980s, a number of roads were constructed over peat bogs in northern Minnesota and in northern Wisconsin using sawmill-generated woodwastes, wood chips, and chunkwood (*4*). Chunkwood is a large blocky material that is manufactured on site using a prototype chunker machine developed by the USDA Forest Service (*5*). Wood chips are a small, thin material that can also be produced on site with a chipper. Field observations indicated that chunkwood had better potential for road building than sawmill woodwastes or wood chips. The chunkwood seemed to have less compressibility than the other materials and seemed to hold up much better under traffic when there was no gravel cover material. With a gravel cover over the wood particles, there seemed to be little difference in the behavior and suitability of the various materials.

The wood chip fills that were constructed in 1983 in northern Minnesota and Wisconsin are still functioning as designed. The wood chip fill seems intact and can still carry normal traffic. It has not settled into the weak muskeg soils and still appears to be highly permeable. In 1986 it was suggested that chunkwood would make a better road than wood chips. Chunkwood and wood chips were comparable in cost to produce.

It was recognized that before wood particles, such as sawmill woodwaste, chunkwood, and wood chips, would be accepted as an engineered road construction material, further testing of the mechanical properties of wood particles was needed. A contract was awarded to Michigan Technological University to conduct a series of laboratory and field tests on the engineering properties of chunkwood (*6*). Field tests indicated that chunkwood did not compact excessively under traffic. Maintenance was minimized where the traffic had enough road width to off-track rather than follow the same path with each pass. The rut depth tended to accumulate to about a 76-mm (3-in.) depth after 20 to 30 passes of a loaded truck with mounds of 51 to 76 mm (2 to 3 in.) high accumulating parallel to the ruts. The maximum measured rut depth after 200 passes of loaded trucks was about 18 cm (7 in.). The ruts were corrected by allowing the trucks to off-track for seven or eight additional passes. The use of a geotextile and the addition of sand or gravel to the surface of the chunkwood greatly increased the stiffness of the roads. It was observed that just enough sand to fill the chunkwood voids would provide the best roadway (*7*).

Laboratory tests were conducted on the chunkwood material. The tests indicated that a permeability of 0.4 km/hr (20 ft/min) should be expected, with a maximum of about 2 km/hr (120 ft/min) (*6*). Field observations confirmed that the material had a very high degree of permeability. Laboratory tests also showed that chunkwood was weaker when the moisture content was high (*7*) and was variable with wood species. A reasonable estimate for future design is a compacted moist unit weight of 640 kg/m^3 (40 lb/ft^3). The Mohr-Coulomb strength law was found to be valid with a cohesion intercept of 14 kPa (2 psi) and an angle of friction of 37 degrees (*6*).

Chunkwood roads have also been constructed in Mississippi, Louisiana, Oregon, Alaska, and several locations in British Columbia, Canada (*5,8*). The major problem with using chunkwood as a road building material is in the production of the chunkwood material. Unfortunately, the woodchunker was developed as a prototype, not as a production machine. To further develop the potential of chunkwood roads, a first-generation production machine will have to be built.

Nemo Point

The Nemo Point demonstration project was implemented because of a number of factors. One factor was the availability of suitable blasted rock borrow and the high cost of hauling when suitable sources could not be found near the project area. Most roads require a depth of 61 to 122 cm (2 to 4 ft) of blasted rock borrow, which combined with hauling is the most expensive component of road construction. Even when rock sources are readily available in the area, road construction costs can be quite high. The cost of single-lane road construction in Alaska is \$86,956 to \$111,801 per km (\$140,000 to \$180,000 per mile), not including major culverts or bridges. This combined with industry's need to dispose of large quantities of sawmill-generated woodwaste provided an opportunity to study the cost-effectiveness and the suitability of these materials for road construction.

A cooperative agreement was made between the USDA Forest Service and the Alaska Pulp Corporation sawmill in Wrangell, Alaska, to construct the 4-km (2.8-mi) Nemo Point demonstration project. Support for the project was to be received from the Alaska Department of Environmental Conservation provided that a water-quality monitoring program was included.

Several large-capacity end dump trucks were used to haul the material to staging areas as construction progressed. Smaller trucks, 8 m^3 (10 yd^3) in capacity, were used to move the material to the construction heading. The material was spread with a Caterpillar D7G bulldozer. Two staging areas were used to stockpile material on site: one was at the beginning of the project; the other 1 km (0.5 miles) from the beginning.

Layer construction was used on the project. Woodwaste fills were constructed in lifts of approximately 51 cm (20 in.). As construction progressed, additional compaction of the layers was achieved through the dump truck traffic. Figure 1 shows the wood fiber embankment before surfacing. Once the road was constructed to subgrade, the aggregate surfacing was placed. For the nontest section areas, a 61-cm (2-ft) depth of shot rock was placed followed by a driving surface of 10 cm (4 in.) of crushed aggregate. Figure 2 shows a typical cross section of the road. Figure 3 shows the finished road.

One of the major construction problems was the lack of compaction on the outside edge of the embankments. Equipment working on the outside edge caused failures, the result of which was that the equipment rolled down the oversteep (1:1) fill slopes. The material naturally stood in a 3/4:1 slope. By layer compacting, a fill slope between 1:1 and $1^1/_4$:1 was achieved.

Engineering Performance Study

The purpose of this study was to determine the engineering performance characteristics of sawmill-generated woodwaste and to establish some general design guidelines for future use. The study was based on the field performance and a laboratory evaluation of the material.

Field Performance

Experiment Design

The field study was designed to evaluate the performance characteristics of the sawmill-generated woodwaste material under various subgrade conditions and surface types. The main emphasis was on the road stiffness and rutting potential. The road stiffness was de-

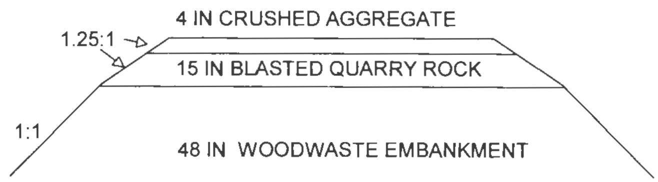


FIGURE 2 Typical cross section of nontest site.

termined using a falling weight deflectometer (FWD) to measure surface deflections. The rutting potential was determined by measuring rut depths after various numbers of truck passes over each section.

Test Section Design

The test sections were designed using a matrix approach to incorporate two thicknesses of woodwaste and two thicknesses of both shot rock and crushed aggregate surfacing. This approach was used for muskeg and non-muskeg subgrade conditions. Table 1 shows the matrix for the two subgrade conditions in combination with the treatments applied to the test sections.

A geotextile was used on half of each test section and was placed between the woodwaste and the aggregate surfacing. Figure 4 shows a typical cross section of the test sites using the geotextile. The geotextile was placed to study the effects on road stiffness and rutting of having a separation layer. The fabric used was a Nicolan style 1120N, a 100 percent polypropylene, nonwoven, needle-punched fabric.

Rutting Potential

For each section, rut measurements were taken on the woodwaste before surfacing. The rutting measurements

FIGURE 1 Unsurfaced woodwaste road.

FIGURE 3 Finished woodwaste road.

TABLE 1 Test Section Design Matrix

WOODWASTE - MUSKEG SUBGRADE CONDITIONS		
AGGREGREGATE	THIN SECTION(30")	THICK SECTIONS(48")
CRUSHED		
THIN(6")	#6	#7
THICK(12")	#5	Future
SHOT ROCK		
THIN(15")	#2	Future
THICK(24")	Future	Future
WOODWASTE - NON-MUSKEG SUBGRADE CONDITIONS		
AGGREGREGATE	THIN SECTION(30")	THICK SECTIONS(48")
CRUSHED		
THIN(6")	#3	Future
THICK(12")		
SHOT ROCK		
THIN(15")	#4	#1
THICK(24")		

on the unsurfaced woodwaste give an indication of the rutting potential of the material. The goal of the rutting study is to correlate rut depth to 80-kN (18-kip) equivalent single axle loads (ESALs).

Rut depths were calculated by measuring the transverse surface profile in relation to a predefined datum. The first measurement is a control that measures the profile in the unrutted condition after blading of the surface. Additional data were collected after various round trips of the dump trucks. The number of round-trip truck passes were converted to an 80-kN (18-kip) ESAL using a USDA Forest Service system based on axle weights and tire pressure (*9*).

Road Stiffness

A Dynatest FWD supplied by the Alaska Department of Transportation was used to measure surface deflections in the test sections. The FWD arrived on site set up for standard pavement applications. Because of the nature of the material and several problems encountered during testing, it was necessary to adjust the loads and drop height to generate accurate information. These problems were a bounce-back or drift effect that caused a vibration in the sensors, and the accurate range of the sensors was insufficient with higher loads. A load of approximately 12 kN (2,600 lbf) was determined to be the highest possible load that would generate accurate data. To achieve these low loadings, it was necessary to reduce the weight configuration to the lowest level and vary the drop height to adjust the final loads at the road surface. Because of the uneven nature of gravel surfaces, sensor location is very important. At many points along the road, a very thin film of sand was required to level the area under the load plate and around the sensors. If the sensors are on a rock or in a depression, the readings are not accurate.

Discussion of Results

General Observations

Before it was surfaced with aggregate, the woodwaste material performed adequately as a driving surface for the construction traffic on the project. It provided a very

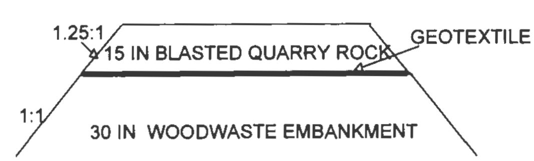


FIGURE 4 Typical cross section of test site with geotextile.

quiet and soft ride as well as good traction for administrative traffic such as pickups and Suburbans.

A woodwaste material depth of 46 cm (18 in.) was able to carry construction traffic in the muskeg areas, but did not seem to provide a significantly stable base for extended use or for supporting a surface layer. Depths greater than 61 cm (24 in.) performed well.

The woodwaste surface tended to produce a low-frequency washboard effect in certain areas. These areas were on grades greater than 12 percent and on either side of any hard spots in the road. The washboard on steep grades seemed to be caused by the driver's need to brake in the favorable direction. The other problem area was around culvert installations and along areas of solid rock excavation. Because these areas are significantly more rigid than solid woodwaste fills, they cause traffic to bounce and they produce washboarding.

The woodwaste material compacts well under construction equipment, but very little compaction occurs in the top 4 to 6 in. This layer seems to stay in a "fluffy" condition and floats over the surface under the influence of the truck traffic. This characteristic is very important in considering the rutting potential of the wood fibers without a surface material. One benefit of this characteristic is that it allows the material to be self-healing if drivers vary the wheel paths between trips.

The types of woodwaste materials used in the embankment made significant differences in the results of the analysis, especially the rutting potential of the woodwaste material. Three types of woodwaste materials were used: sawdust, planer chips, and bark fibers. Each of these materials behaves differently. Sawdust does not seem to perform as well as the other fibers. Its particles are very small and tend to break down under traffic loading. When it is predominant in the section, deeper rutting tends to occur. Planer chips do not compact very well. These are very thin fibers of various shapes and sizes that tend to resist compaction. Bark fibers are long thin fibers that intertwine and form a well-compacted layer. The woodwaste material seemed to perform the best when placed with a good mixture of particle types.

Rutting Results

The objective of the analysis was to determine the relationship, if any, between the rut depth and 80-kN (18-kip) ESALs. Simple linear regression was used to obtain the results presented below.

The regression analysis determined the relationship between rut depth and ESALs to be rut depth = 1.749 + 0.012*(ESALs), with a resulting two-sided p-value of 0.00107. The standard errors for the intercept and slope are 0.197 and 0.0035, respectively. The 95 percent confidence interval for the slope parameter is 0.00524 to 0.0195. The small p-value shows evidence of an association between these variables. A lack-of-fit analysis also suggests a relationship between the variables (p-value = 0.534).

Although the results of the analysis suggest that there is an association between the variables, only a small percentage of the variation is accounted for by the independent variable ESALs, $R^2 = 21.79$. For this type of application, it may be enough to adequately design for rutting. However, there are several factors that may account for additional variability. These are the moisture content of the material, the type of wood fibers that make up a majority of the test section, and the fluffy nature of the top few inches of the unsurfaced layer. Further analysis should be done on the surfaced woodwaste to determine how the various surface materials affect the rutting potential.

Road Stiffness Results

The FWD data were used to evaluate the resilient modulus of the woodwaste layer. The deflections for each test point were normalized to a loading of 9 kN (2,000 lbf) and were analyzed using the computer program Bousdef 2 (*10*). A statistical analysis of the data was done using a one-way analysis of variance.

The average resilient modulus for all points was 14 kPa (2,041 psi) with a standard error of 1 kPa (189 psi). Several variables were analyzed for their effect on the stiffness of the woodwaste layer. An analysis of variance was done to compare modulus values with respect to woodwaste depth, aggregate surface depth, the use of a geotextile, and the subgrade type.

The first test compared the modulus values according to the depth of the woodwaste layer. This test shows no statistical difference in the mean modulus values of the two groups (p-value = 0.3537). The second analysis compared the modulus values according to the depth of surface aggregate. Three surface depths were used on the project: 15 cm (6 in.) and 30 cm (12 in.) of crushed aggregate and 38 cm (15 in.) of shot rock. The test shows a statistical difference between the groups (p-value = 0.00). A comparison of the use of geotextile shows no statistical difference between the fabric and no fabric groups. Many types of geotextiles are available on the market. A difference type of fabric may produce a different effect on the stiffness of the layer. Table 2 shows the results of these tests.

The analysis of variance shows that the aggregate surface thickness has the most potential influence on the stiffness of the woodwaste layer. The thicker surfaces yielded higher average wood-layer modulus values. The thicker surface layers appear to control the deflections, producing lower deflections and yielding higher resilient modulus values for the woodwaste layer. Part of the

TABLE 2 Analysis of Variance Test Results for FWD Deflection Data

WOODWASTE DEPTH	0.76 m (30 in.)	1.22 m (48 in.)	
MEAN	14.9 KPa (2162 psi)	12.2 KPa (1769 psi)	
STANDARD ERROR	1.67 KPa (243 psi)	1.90 KPa (276 psi)	
AGGREGATE DEPTH	**0.15 m (6 in.)**	**0.305 m (12 in.)**	**4.6 cm (15 in.)**
MEAN	9.05 KPa (1313 psi)	25.09 KPa (3642 psi)	15.25 KPa (2214 psi)
STANDARD ERROR	1.17 KPa (170 psi)	3.84 KPa (558 psi)	1.46 KPa (212 psi)
GEOTEXTILE USE	**GEOTEXTILE**	**NO GEOTEXTILE**	
MEAN	16 MPa (2294 psi)	12 MPa (1774 psi)	
STANDARD ERROR	2 MPa (300 psi)	1MPa (216 psi)	

reason for this is that the thicker layers produce a higher static load, compressing the woodwaste and causing it to become more stiff. The difference between the crushed and shot rock surfaces is likely because of the material gradation. The denser gradation of the crushed aggregate provides better particle interlock, allowing the surface to spread the load over a wide area. The shot rock contains a large volume of voids and consists of predominantly large particle sizes, which may cause shearing at the aggregate wood fiber interface.

Laboratory Analysis

Laboratory Procedures

The laboratory test procedures were designed to complement the field performance measurements taken on the project. The main emphasis was on the moisture density relationship, particle size analysis, and on the resilient modulus of the wood fibers. Standard test procedures for subgrade soils and aggregate were used to determine the engineering properties of the material in accordance with AASHTO test procedures.

Discussion of Results

The moisture density curve is very flat over a wide range of moisture contents. The densities range from 290 kg/m^3 (18.1 lb/ft^3) at 105 percent moisture content to a maximum dry density of 293 kg/m^3 (18.3 lb/ft^3) at an optimum moisture content of 166 percent. These values are very similar to those reported by Nelson and Allen (*12*). They report a maximum dry density of 341 kg/m^3 (21.3 lb/ft^3) at a moisture content of 175 percent for cedar hog fuel. The slight difference may be attributed to differences in wood species and particle size.

A sieve analysis was conducted to determine the particle size distribution of the woodwaste material. The results are based on the average percent passing of five separate tests. The nominal maximum particle size is 25 mm (1 in.). Field observations suggest that this may be as large as 15 cm (6 in.) The analysis shows that the woodwaste is a relatively well-graded or densely graded material. It contains a broad range of particle sizes with a higher percentage of material in the No. 4 to No. 40 size classification.

The stability of typical base and subbase materials such as soil-aggregate mixes depends on several factors: particle size distribution, shape, relative density, and internal friction (*12*). Several of these factors may also be applicable to woodwaste. The elongated shape produces an interlocked structure when compacted and the gradation contains enough fine material to fill the voids without "floating" the larger particles in the mixture. These factors should contribute to the stability, shear resistance, and load distribution characteristics of the material.

Laboratory resilient modulus tests were conducted to verify back-calculated modulus values obtained in the field with an FWD. The laboratory resilient modulus results were analyzed in two ways (*11*). The first analysis treated the woodwaste modulus as a function of bulk stress as is done with coarse-grained materials (*12*). The second analysis treated the modulus as a function of deviator stress as is done with fine-grained materials (*12*). In both analyses, the woodwaste modulus did not exhibit the typical responses associated with coarse- or fine-grained materials (*12*).

The bulk stress analysis relationship,

$$Mr = K1 \times \theta^{K2} \tag{1}$$

was used to express the resilient modulus, where $K1$ and $K2$ are regression constants and θ is the bulk stress.

The modulus values for each specimen were plotted versus bulk stress on a log-log scale using Equation 1 to fit the data. Figure 5 shows the woodwaste moduli responses for Samples 1 to 3. The results do not show very good correlation with accepted methods.

The second analysis looked at the responses of modulus as a function of deviator stress. Figure 6 shows the typical relationship found in the analysis. The results show that there does not seem to be a standard relationship between modulus and deviator stress for these data. The samples have a wide variation in curve shapes. The most typical seems to be a trend toward decreasing modulus with increasing deviator stress.

Average modulus values do not represent expected values very well because of the variability found in the analysis. Laboratory modulus values range from 5 to 34 MPa (700 to 5,000 psi). The moduli do not seem to have a definite pattern with respect to stress condition, moisture content, or density at this time. More laboratory analysis should be done to better define the resilient modulus of sawmill-generated woodwaste.

Future Studies

R-Value Study

One of the engineering properties of small-wood-particle road construction that is thought to be substantially different from conventional gravel-and-soil construction is the potential insulating effect. In cold regions, this can have a significant positive influence on the potential for frost heave, the loss of strength as a result of spring thaw, and protection of permafrost from thaw conditions. Empirical observations in Minnesota and Wisconsin indicate that there is a very significant insulating effect from the materials, but no tests have been conducted to determine the insulating properties.

The objective of the future study is to determine the thermal resistance (R-value) of selected samples of chunkwood using a modified hot-box assembly. Once the laboratory tests have been conducted to determine the R-value for chunkwood, it is anticipated that field tests will be conducted. The tests should be in an area with enough freezing days to show the difference between traditional soil-gravel construction and chunkwood-gravel construction. With the results of the field tests, the suitability of chunkwood for construction over permafrost and frost-susceptible soils will be largely known. In severe Arctic conditions, up to 2 m (6 ft) of gravel may be needed to protect the permafrost from thawing under the road. If a foot or two of wood could reduce the amount of gravel needed, a substantial savings is possible.

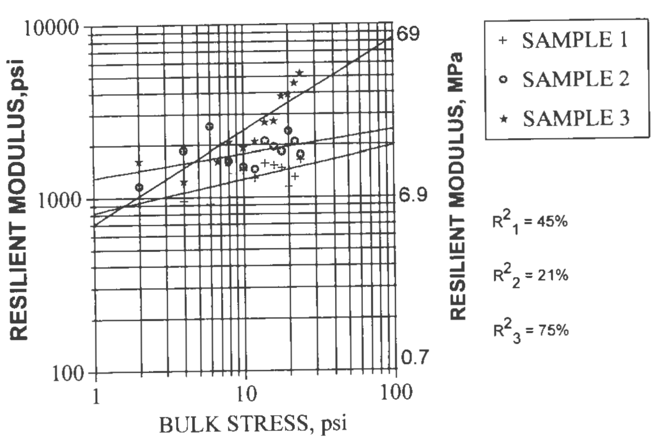


FIGURE 5 Resilient modulus versus bulk stress, Samples 1–3.

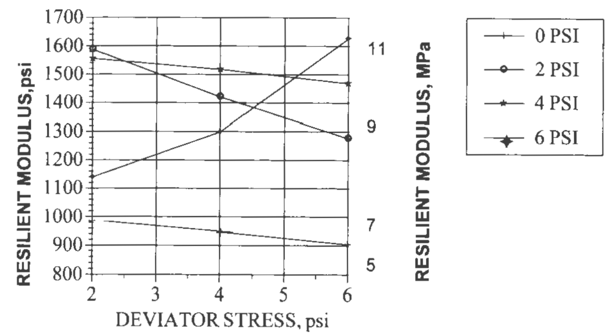


FIGURE 6 Resilient modulus versus deviator stress, Sample 1.

Wood Preservative Study

The long-term use of small wood particles as construction material in temperate and warmer climate depends on a low-cost, low-toxicity wood preservative. Another problem that needs to be solved is the potential of the material to combust and burn. A literature review indicates that the most practical solution is borate. It is very low cost, serves as an effective fireproofing material, and has a very low toxicity. Its major defect is that it tends to leach out with repeated wetting-drying cycles. The use of geotextiles to prevent leaching should be considered. The potential to use borate as a low-cost wood preservative seems to be an idea that should also be tested.

Summary

All types of wood fibers have a long history of use in road construction across the United States and British

Columbia. Wood particles have been used in highways and low-volume roads to reduce landslide potential and to cross settlement-sensitive soils such as muskeg. Many types of wood particles are being used: brush, construction slash, chunkwood, and sawmill-generated woodwaste such as bark, sawdust, and planer shavings.

The USDA Forest Service recently constructed 4 km (2.8 mi) of forest access road in Wrangell, Alaska, using sawmill woodwaste supplied by the Alaska Pulp Corporation under a cooperative agreement. This project was conducted to study the economic feasibility and the material's suitability for use in the area. As part of the study, an evaluation of the engineering performance characteristics was undertaken to develop some guidelines for future use. The study evaluated the rutting potential and road stiffness in a series of field and laboratory analyses.

Preliminary results of the engineering performance study show that sawmill-generated woodwaste is a suitable construction material for use in southeast Alaska. These materials are adequate as embankment materials or as a driving surface. When used as embankment material, a woodwaste depth of 76 cm (30 in.) seems to be the minimum depth required to carry construction traffic and subsequent heavy-duty traffic for the test sites. The woodwaste materials perform best when surfaced with 30 cm (12 in.) of crushed aggregate or 38 cm (15 in.) of shot rock. If the road is intended for light-duty recreational or administrative traffic, 15 cm (6 in.) of crushed aggregate may be adequate.

As a driving surface, woodwaste provides a soft, quiet ride. Frequently scheduled maintenance is required to repair rutting and low-frequency washboarding that is prone to develop. These can be easily corrected by blading with a standard motor grader or bulldozer. Rutting of up to 7 cm (3 in.) tends to occur within 20 to 30 round-trip truck passes. If the road has sufficient width for off-tracking, the wood fibers tend to be self-healing with respect to rutting.

Initial laboratory analysis shows that over a wide range in moisture contents, woodwaste dry density is not significantly variable. This means that layer placement and compaction with construction equipment is a viable construction technique. It is suggested that layer depths of 51 to 61 cm (20 to 24 in.) be used to maximize density. If the layers are to remain for several days before placing the next layer, occasional patching may be necessary to compensate for soft spots and material loss.

Potential future studies include an *R*-value and wood preservative study. The *R*-value study would determine the insulating properties of wood fibers for use in cold regions to cross permafrost. A wood preservative study would study the use of nontoxic chemicals and geotextiles for extending wood fiber life cycles. Both of these studies could have a large influence on the future use of wood fiber materials in road construction.

ACKNOWLEDGMENTS

The authors would like to thank the USDA Forest Service, Region 10, Tongass National Forest, and William Hartsog, James Schaeffer, and Bruce Brunette, in particular, for their work in making this project possible. They would also like to thank Dave Sterley and Wayne Hoyt of the Alaska Department of Transportation for making the falling weight deflectometer available for use on this project. Special thanks go to the Alaska Pulp Corporation in Wrangell, Alaska, for working with the authors to construct the test sections as needed and for their patience while the testing was in progress.

REFERENCES

1. Dittmer, M. H. Efforts to Reduce Construction Costs of Logging Roads in Muskeg and Wet Soils in Southeast Alaska. In *Transportation Research Record 1106*, Vol. 2, TRB, National Research Council, Washington, D.C., 1987, pp. 267–273.
2. Bowman, J. K., R. B. Lidell, and G. B. Schulze. The Use of Wood Chips in Low-Volume Road Construction in the Great Lake States. In *Transportation Research Record 1106*, Vol. 1, TRB, National Research Council, Washington, D.C., 1987, pp. 47–58.
3. Coulter, T. S. Woodwaste as a Lightweight Fill in Highway Construction. In *Proc., 28th Conference of the Western Association of Canadian Highway Officials*, 1975, pp. 1–12.
4. Hodek, R. J., R. A. Arola, J. K. Bowman, and G. B. Schulze. Forest Roads Built with Chunkwood. In *Proc., International Energy Agency*, Task III/Activities 6 and 7, Research Notes no. 134/1988, Vol. II, Swedish University of Agricultural Sciences, Garpenburg, 1988, pp. 128–149.
5. Karsky, D. *Chunkwood Roads*. 9371-2818-MTDC. USDA Forest Service Technology and Development Program, Missoula, Mont., 1993.
6. Hodek, R. J., and L. W. Shook. *Evaluation of Chunkwood as a Low-Volume Road Building Material*, Final Report, USDA Forest Service, Chequamegon NF, Michigan Technological University, Houghton, 1986.
7. Shook, L. W. An Investigation of *Selected Geotechnical Engineering Properties of Chunkwood*. Michigan Technological University, Houghton, 1988.
8. Arola, R., J. Bowman, R. Hodek, and G. Schultz. Wooden Roads Revisited: A Substitute for Gravel Roads. *Forest Industries*, Vol. 117, No. 5, June 1990.
9. Whitcomb, W. G., M. T. Yapp, and M. Myers. *Aggregate Surfacing Design Guide*. USDA Forest Service, August 1991, pp. 22–29.

10. Zhou, H., R. G. Hicks, and C. A. Bell. *Bousdef: A Back-calculation Program for Determining Moduli of a Pavement Structure.* Oregon State University, Corvallis, 1988.
11. Hicks, R. G., and T. S. Vinson. *Resilient Modulus Repeated Load Test System: Users Manual.* H and V Materials Research and Development, Inc., July 1989.
12. Nelson, D. S., and W. L. Allen. *Sawdust as a Lightweight Fill Material.* Federal Highway Administration, PB-233 754, 1974, p. 7.

Waste Products in Highway Construction

Chunhua Han and Ann M. Johnson, *Braun Intertec Corporation*

The use of waste materials and their products for highway construction is discussed. The general legislation, local liability, and research projects related to waste materials are outlined. The waste materials and products presented include waste paving, industrial ash, taconite tailings, waste tire rubber and products, building rubble, incinerator ash and products, waste glass, waste shingle and products, waste plastics and products, and slag. For each waste category, the legislation and restrictions, material properties, construction and application, field performance, and recycling at the end of service life, if available, are discussed. In addition, procedures for evaluation of and selection from waste alternatives are presented. Results from a survey sent to Minnesota city and county agencies are presented summarizing current practices in waste reuse for highway construction.

Highway construction projects depend on an adequate supply of aggregate and mineral filler. The demand for such filler used in highway construction has increased dramatically, especially where aggregate sources have been depleted, the quality of available aggregate is at a low level, or aggregate cannot be obtained because of mining restrictions, environmental protection regulations, or appreciating land values.

In contrast, enormous quantities of domestic, industrial, and mining waste are generated annually in the United States. An extensive effort to reuse wastes in highway construction has been made by researchers and engineers for almost a century; many reports and findings have been produced in this area (*1–55*). At present, there are seven reasons for an agency to consider the reuse of wastes:

- Shortage of aggregates,
- High cost of waste disposal,
- Commitment to the environment,
- Availability of virgin and waste materials,
- Local availability,
- Political pressure, and
- Environmental safety.

Four issues are fundamental in determining the appropriateness of using recycled waste materials in highway construction:

- Cost-effectiveness,
- Performance,
- Availability, and
- Prevailing political climate.

The high cost of processing wastes for reuse and the uncertainty of their performance and durability require that a better justification of their use be provided. This paper establishes an inventory of waste sources and provides technical definitions and sources of waste products. Based primarily on experience with their use in Minnesota, an evaluation of waste materials is given, along with the field performance of roads built with them. This report also summarizes survey results regarding the use of waste materials in Minnesota highway construction, based on responses from city and county engineers.

Although waste products have been used in all sizes of highways and roadways, a study of their application is particularly important in low-volume roadways. Many of the products outlined in this paper have not been tested adequately to define their field performance. By placing them in low-volume roadways, their properties can be determined and guidelines for their use can be refined.

A full report on waste materials in highway construction giving complete descriptions and properties of waste materials is available from the Minnesota Local Road Research Board (*56*). This paper serves as an overview of practices and materials available in Minnesota. Table 1 presents waste materials frequently used in Minnesota.

Waste Material Use and Treatment

Without modification in properties or addition of ingredients, a waste can be used as a mineral filler, additive, or aggregate in highway construction. Many wastes are potential admixtures, particularly when processed. The processed wastes generally can be obtained from a recycling or processing facility, and some have become commercial products. If the properties of a waste do not conflict with and are compatible with the properties of asphalt or portland cement, it is a potential aggregate for asphalt or concrete mix. A waste may also be directly placed as a base course if it satisfies the base material specifications and leachate requirements.

Proper evaluation of a specific waste material requires a basic knowledge of its physical and chemical characteristics. These properties must be obtained in order to meet the requirements for construction materials and the environmental protection regulations. Detailed material properties are available in the report by Han and Johnson (*56*). Table 2 gives waste products obtained from or reused in highway construction. Note that many of the materials have several uses.

Initial Screening

The initial screening of a waste material for its suitability in construction is a crucial start that leads to a cost-effective evaluation. The screening is based on various minimum criteria constrained by environmental regulations, construction requirements, geographic limitations, quantity of materials available, and local conditions.

TABLE 1 Waste Materials Frequently Used in Minnesota

Waste	Source	Condition	Aggregate
Flyash	Coal burning power plants	dust	Yes
Bottom ash	Coal burning power plants	fine sand	Yes
Boiler slag	Coal burning power plants	gravel	Yes
Steel slag	Iron and steel production plants	coarse	Yes
Bituminous coal refuse	Bituminous coal mines	fine and coarse	Yes
Dredge spoil	Navigable waterways	slurry	Yes
Taconite tailings	Taconite mines	slurry and fine	Yes
Building rubble	Demolition	coarse	Yes
Incinerator residue	Municipal incinerator	ash	Yes
Rubber tires	Automobile and truck tires	coarse and fine	Yes
Sewage sludge	Sewage treatment plants	slurry and ash	Yes
Glass	Container glass	coarse	Yes
Pyrolysis residue	Pyrolysis operations	char	Yes
Reclaimed paving waste	Highway constructions	coarse	Yes
Wood chips and sawdust	Logging-chipping operations	coarse	Yes
Battery casings	Automobile batteries	coarse	Yes

TABLE 2 Waste Products in Highway Construction

Waste	Description	Treatment	Use	Performance
Waste Paving Material				
Crushed Concrete	mix of stone, dirt, wood, brick, organic, & concrete	crushed, impurities removed	concrete mix aggregate; base aggregate	excellent
Pulverized Bituminous	mix of bituminous materials & aggregate	crushed	cold in-place recycling; as aggregate in washout areas	excellent
Industrial Material				
Flyash	finely divided residue w/pozzolanic properties	added to concrete to form flyash concrete (FAC); added to aggregate base for stabilization	additive; embankment or subgrade fill	in concrete, improved workability; reduces bleeding
Bottom Ash	finely divided residue from electric power generation		additive; embankment or subgrade fill	good
Mineral Material				
Iron Ore & Taconite Tailings	obtained from processing or pelletizing of iron ore & taconite	none	bituminous mix aggregate	suitable for thin overlays; requires 1-2% more AC than conventional mixes
Domestic Material				
Waste Tires	mechanically processed to achieve size & void reduction; may be left whole	shredded, chipped, or ground into crumb rubber additive (CRA); may be used whole	additive; embankment or subgrade fill; also used as safety feature in protective crash cushions	non-biodegradable; more durable than wood chips in fill; durability of CRA pavements still unresolved
Building Rubble	mix of concrete, plaster, steel, wood, brick, piping, AC, glass	must be crushed & sized, impurities removed	base or subgrade aggregate	good
Incinerated Sewage Sludge Ash	after primary treatment a liquid w/solids content of 5-10%	incinerated and incorporated into mixtures such as lime-flyash sulfate	additive; aggregate	adequate strength for road embankment construction
Waste Glass	obtained from roadside recycling	crushed, resulting in flat, elongated particles w/smooth surfaces & no porosity	base and subgrade aggregate additive; bituminous mix aggregate	has been show to improve thermal characteristics of paving mixtures
Incinerated Municipal Sludge	ash waste or incinerator ash residue; bottom ash consists of slag, glass, rocks, metals, and unbound organic matter	chemically fixed, forming treated ash pellets (TAP)	base, subbase, and pavement aggregates	TAP meets MnDOT specifications
Waste Shingles	obtained from roofing manufacturers	shingles ground for their aggregate and asphalt cement	additive in bituminous mixes	satisfactory; research continuing
Municipal Solid Waste Plastics	obtained from roadside recycling	melt-extruded into post & board shapes that can be applied to guardrail & fenceposts	safety features; fence, guardrail	flexural stress can be higher than concrete
Steel Slag	must be aged 6-7 months to allow complete expansion	none	base and subbase aggregate; concrete mix aggregate	good
Wood Chips and Sawdust	obtained from municipal solid waste sources	placed over polymer geogrid, spread, & compacted	lightweight fill	good

TABLE 3 Technical Feasibility for Aggregate Use in Base Courses

Rank	Class 1	Class 2	Class 3	Class 4
1	Flyash	Slate Mining Waste	Phosphate Slime	Iron Ore Tailings
2	Bottom Ash	Steel Slag	Rubber Tires	Sewage Sludge
3	Boiler Slag	Anthracite Coal Refuse	Foundry Waste	
4	Shingle Scrap	Taconite Tailings	Dredge Spoils	
5	Zinc Smelter Waste	Lead-Zinc Tailings	Bituminous Coal Refuse	
6	Gold Mining Waste	Phosphate Slag	Battery Casings	
7	Paving Waste	Incinerator Residue	Sulfate Sludge	
8	Waste Glass	Feldspar Tailings	Scrubber Sludge	
9	Blast Furnace Slag	Building Rubble		

The minimum environmental criterion is that a waste candidate must be nonhazardous. A waste product should be identified following the standard procedures in order to determine if it is hazardous. Detailed criteria for identifying hazardous waste can be found in *Minnesota Rules*, Parts 7045.0120 to 7045.0135 (*19*).

Material requirements for highway construction are the basic criteria for selecting waste materials. The potential waste replacements for cement or aggregate should satisfy the corresponding construction requirements. The waste material must be located within a reasonable geographic distance from a construction site or transportation costs will be prohibitive; 40 to 50 mi is considered a maximum economic hauling distance for truck transport and 100 mi for rail transport.

Technical Evaluation

The technical feasibility of using a waste in construction can be evaluated on the basis of its technical properties, including physical, mechanical, chemical, thermal, and optical properties related to specific highway applications. A simple evaluation system can be established by listing technical properties of waste candidates relevant to the application considered. In evaluating the number of properties relevant to the application, waste candidates are classified in the following manner: the more relevant properties a waste possesses, the more potential it has, and the higher it will be ranked. A four-class technical evaluation system could be used as follows:

- Class 1: wastes that have the highest potential for use and require a minimum of processing before use;
- Class 2: wastes that have a relatively high potential and require more extensive processing such as pelletizing and sintering;
- Class 3: wastes that have a relatively low potential for use and may require a formidable amount of processing, that may have some outstanding undesirable physical properties, and that may have rather nonuniform characteristics; and
- Class 4: wastes that have little or no potential and at best might be used in small amounts as filler or in very specialized applications.

A number of waste materials were evaluated for their potential use as aggregates using the four-class system, as shown in Tables 3 through 5. The wastes listed are also ranked in each class.

Overview of Current Statewide Practice

To obtain information on the current practices of waste reuse in Minnesota highway construction, a question-

TABLE 4 Technical Feasibility for Aggregate Use in Bituminous Mix

Rank	Class 1	Class 2	Class 3	Class 4
1	Flyash	Anthracite Coal Refuse	Rubber Tires	Sewage Sludge
2	Bottom Ash	Lead-Zinc Tailings	Bituminous Coal Refuse	
3	Shingle Scrap	Building Rubble	Foundry Waste	
4	Boiler Slag	Steel Slag	Battery Casings	
5	Zinc Smelter Waste	Feldspar Tailings	Iron Ore Tailings	
6	Gold Mining Waste	Copper Tailings	Slate Mining Waste	
7	Paving Waste	Phosphate Slag	Dredge Spoils	
8	Blast Furnace Slag	Phosphate Slime	Sulfate Sludge	
9	Waste glass	Incinerator Residue	Scrubber Sludge	

TABLE 5 Technical Feasibility for Aggregate Use in Concrete Mix

Rank	Class 1	Class 2	Class 3	Class 4
1	Flyash	Feldspar Tailings	Bituminous Coal Refuse	Sewage Sludge
2	Bottom Ash	Taconite Tailings	Building Rubble	Waste glass
3	Shingle Scrap	Anthracite Coal Refuse	Iron Ore Tailings	
4	Boiler Slag	Steel Slag	Zinc Smelter Waste	
5	Zinc Smelter Waste	Foundry Waste	Rubber Tires	
6	Gold Mining Waste	Incinerator Residue	Slate Mining Waste	
7	Paving Waste	Phosphate Slime	Dredge Spoils	
8	Blast Furnace Slag	Copper Tailings	Battery Casings	
9	Waste Glass	Lead-Zinc Tailings	Sulfate Sludge	
10			Scrubber Sludge	

naire was developed and distributed to all Minnesota cities and counties. Of the 198 questionnaires distributed, 79 cities and counties responded (40 percent). Beside providing answers to the specific questions, respondents also sent information concerning their own use of various waste materials in highway construction. The survey helped determine the latest trends, applications, and experiences in the use of waste materials (C. Han, unpublished data, 1993).

Among responding agencies, 39 had experience with the reuse of wastes in highway construction, 4 had experience in recycling, 1 is considering the reuse of waste, and 35 had no experience. As shown in Figure 1, many waste materials are being used by agencies, including paving materials with no salvage value, coal fly ash, waste glass, building rubble, coal bottom ash, sewage sludge, rubber tires, asphalt shingle, waste paper, mine tailings, and wood chips.

A total of 14 waste products are in use or are being studied experimentally in a variety of highway applications. Current practice indicates that a large number of respondents use waste paving materials, fly ash, and scrap tires. Most waste materials used were evaluated as at least competitive with the conventional materials. However, the use of steel slag, mine tailings, and scrap tires was considered uneconomical.

CASE STUDIES

Case 1: Shredded Tires, Benton County

Near Rice in Benton County, shredded tires were used as a lightweight fill material for State Aid Highway 21. This road is actually floating over swampy soils. The two-lane highway was originally constructed with a

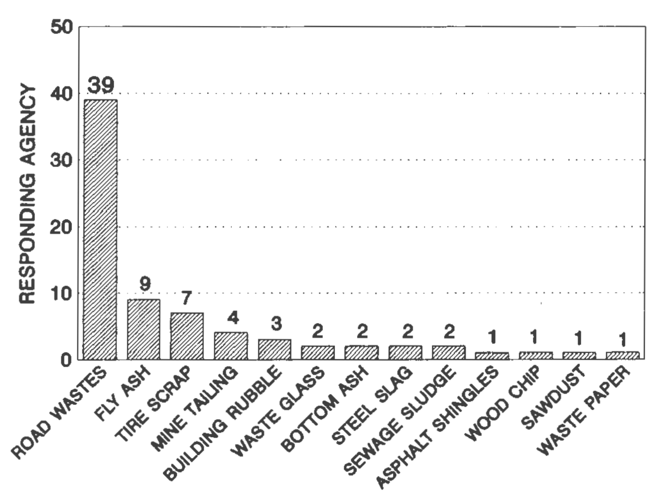


FIGURE 1 Use of waste materials in Minnesota highway construction.

sand-and-gravel subbase and was performing well. Over the years, the surrounding water levels increased to the level of the road. An attempt to raise the roadway with conventional granular fill overloaded the underlying 12-ft layer of peat and muck, causing an embankment failure. After performing a cost-benefit analysis, the county decided to use shredded tires as a lightweight fill material in reconstruction.

Reconstruction on the 250-ft section began in the fall of 1989. The first step was to excavate to a point $^1/_2$ ft above the swamp or marsh level. Next, a geotextile fabric was sewn together and positioned at the bottom of the excavation. Following the fabric, approximately 52,000 shredded tires were deposited in a 2-ft lift to a level of 3.5 ft below the top of the subgrade elevation. The shredded tires were compacted with bulldozers and front-end loaders until no further compaction was detected and were overlaid with another geotextile fabric layer. No moisture content was specified in the compaction process. Granular materials were placed over the fabric, and the fill was compacted using ordinary compaction. Finally, the new subbase and gravel base were constructed, and the roadway was allowed to settle naturally due to overburden for several months without traffic loadings. The bituminous surface was placed the following spring.

To date, the county road has not experienced any significant settlements and the bituminous surface is performing well.

Case 2: Waste Glass, Sibley County

Sibley County, the Office of Waste Management, and the Minnesota Department of Transportation combined efforts in a project to utilize waste glass with low-grade aggregate for better base materials. The mixed base materials were used to rebuild Sibley County Road 6.

Three hundred and thirty tons of mixed glass that were not suitable for normal glass recycling were used. The glass was crushed with a low-grade aggregate to make a Class 5 gravel base, containing approximately 10 percent glass. The introduction of the glass not only reduced the percentages passing the $^3/_8$-in., No. 4, No. 10, and No. 40 sieves as anticipated, but it also increased the portion passing the No. 200 sieve by about 2 percent, which was not anticipated.

The mixed-glass aggregate was placed in a 1,000-ft test section on the 3.7-mi construction project. Three 3-in. lifts were placed topped with a final 4-in. lift of virgin Class 5 aggregate and sealed with a 3-in. bituminous surface. During construction, the surface was exposed to local traffic without incidence of tire puncture or any other apparent problems, and raveling of the surface appeared to be less in the test section constructed with the glass aggregate mixture. Except for more power and downshifting of gears required to place the mix because of greater friction, grading and compaction of the material went without incident.

Preliminary results indicate that low-quality "sandy" aggregate can be enhanced with the introduction of crushed glass, thus increasing the utilization of low-quality aggregate and disposing of an otherwise useless waste material.

Summary

An evaluation based on technical, environmental, and economic factors indicated that waste paving materials, fly ash, incinerator ash, waste shingles, rubber tires, and slag have significant potential to replace portions of conventional highway construction materials. The reuse of these waste products can be realized by a combined effort among agencies involved with waste management, natural source reserves, environmental protection, and highway construction.

Waste recycling and processing provide substitute construction materials as well as secondary waste materials. Specifications and construction procedures are needed for these materials to be applied to highway construction. It must be noted that highways and roadways are a long-term investment that must be both cost-effective and durable. Before widespread and general acceptance can be made of a waste product used in highway construction, it must be evaluated in a pilot project over a long period of time to quantify its actual performance. After pilot projects have been successfully constructed, specifications developed, and suitable long-term evaluations made, these materials can be routinely used. In the future, a complete closed-loop recycling process can be developed moving from product to waste infrastructure.

Waste processing by incinerating or composting also produces more and more secondary wastes. Under controlled construction, these processing residues can be utilized without imposing environmental risk. In this way, controlled disposal and construction are combined into one practice, thereby resulting in a cost-effective alternative to traditional means of road construction.

REFERENCES

1. Marquid, E. L., T. J. Hirsch, and C. E. Buth. *NCHRP Report 157: Crash Cushions of Waste Materials.* TRB, National Research Council, Washington, D.C., 1975, pp. 1–73.
2. Boles, W. F. *Flyash Facts for Highway Engineers.* Demonstration Projects Program, FHWA-DP-59-8. FHWA, U.S. Department of Transportation, 1986, pp. 1–47.

3. Halstead, W. J. *NCHRP Synthesis of Highway Practice 127: Use of Flyash in Concrete.* TRB, National Research Council, Washington, D.C., 1986, pp. 1–27.
4. Faber, J. H. Flyash Utilization—Problems and Prospects. *Proc., 1st Mineral Waste Utilization Symposium,* IIT Research Institute, Chicago, Ill., 1968, pp. 99–107.
5. Brackett, C. E. Availability, Quality, and Present Utilization of Flyash. *Proc., 1st Ash Utilization Symposium,* Pittsburgh, Pa., 1967, pp. 16–36.
6. Smith, R. D. Attack of Glasses by Alkaline Solutions. *Journal of the American Concrete Institute,* Vol. 32, No. 9, 1949, pp. 195–198.
7. Malisch, W. R., D. E. Day, and D. G. Wixson. Use of Waste Glass for Urban Paving. *Proc., 2nd Mineral Waste Utilization Symposium,* IIT Research Institute, Chicago, Ill., 1970, pp. 369–373.
8. Paulsen, G., M. Stroup-Gardiner, and J. Epps. *Roofing Waste in Asphalt Paving Mixtures.* Department of Civil Engineering, University of Nevada, Reno, 1988, pp. 1–44.
9. Piggott, M. R., and R. T. Woodhams. *Recycling of Rubber Tires in Asphalt Paving Materials.* Department of Chemical Engineering and Applied Chemistry, University of Toronto, Canada, 1979, pp. 4–6.
10. Ormsby, W. C., and D. G. Fohs. Use of Waste and By-Products in Highway Construction. In *Transportation Research Record 1288,* TRB, National Research Council, Washington, D.C., 1990, pp. 47–58.
11. Burlingame, M. J. Construction of a Highway on a Sanitary Landfill and Long-Term Performance. In *Transportation Research Record 1031,* TRB, National Research Council, Washington, D.C., 1985, pp. 34–40.
12. Bowman, J. K., R. B. Lidell, and G. B. Schulze. The Use of Wood Chips in Low-Volume Road Construction in the Great Lake States. In *Transportation Research Record 1106,* Vol. 1, TRB, National Research Council, Washington, D.C., 1987, pp. 47–58.
13 Heitzman, M. Design and Construction of Asphalt Paving Materials with Crumb Rubber Modifier. In *Transportation Research Record 1339,* TRB, National Research Council, Washington, D.C., 1992.
14. Stephens, J. E. *Nine-Year Evaluation of Recycled Rubber in Roads—Final Report.* No. JHR 89-183. School of Civil Engineering, University of Connecticut, Storrs, 1989, pp. 1–16.
15. Sommerfeld, J. C. The Use of Flyash in Highway Construction. In *Proc., AMCE Specialty Conference, New Materials and Processes for Street, Highway and Airport Rehabilitation,* ASCE, New York, 1983, pp. 345–355.
16. Klemens, T. L. Processing Waste Roofing for Asphalt Cold Patches. *Highway and Heavy Construction,* April 1991, pp. 30–31.
17. Turgeon, C. M. *Waste Tire and Shingles Scrap/Bituminous Paving Test Sections on the Willard Munger Recreational Trail Gateway Segment.* Interim Report, MN/RD-91/06. Minnesota Portland Cement Association, Department of Natural Resources, and Department of Transportation, 1991, pp. 1–8.
18. Helland, J. K. *A Guide to Minnesota's Major Environmental Laws.* Research Department, Minnesota House of Representatives, St. Paul, 1990, pp. 1–47.
19. *Minnesota Rules, 1991.* The Revisor of Statutes, State of Minnesota, 1991, pp. 6355–6526.
20. Freedman, W. *Hazardous Waste Liability.* Michie Co., Charlottesville, Va., 1987, 664 pp.
21. Gannon, C. R., et al. Recycling Conventional and Rubberized Bituminous Concrete Pavements Using Recycling Agents. *Proc., Association of Asphalt Pavement Technologists,* Vol. 49, 1980, pp. 95–122.
22. *NCHRP Synthesis of Highway Practice 54: Recycling Materials for Highways.* TRB, National Research Council, Washington, D.C., 1978, pp. 1–53.
23. *Characterization of Municipal Solid Waste in the United States: 1990 Update.* EPA/530-SW-90-042. Environmental Protection Agency, 1990, pp. 1–103.
24. Marek, C. R., M. Herrin, C. E. Kesler, and E. J. Barenberg. *NCHRP Report 135: Promising Replacements for Conventional Aggregates for Highway Use.* HRB, National Research Council, Washington, D.C., 1972, 53 pp.
25. Miller, R. H., and R. J. Collins. *NCHRP Report 166: Waste Materials as Potential Replacements for Highway Aggregates.* TRB, National Research Council, Washington, D.C., 1976, 94 pp.
26. Abdun-nur, E. A. Flyash in Concrete—An Evaluation. *HRB Bulletin 284,* HRB, National Research Council, Washington, D.C., 1961, pp. 1–12.
27. *Minnesota Recycling Directory.* Report 830033. MPCA, Minnesota, 1982, pp. E1–E5.
28. *Minnesota Recycling Directory.* Minnesota Waste Management Board, Minnesota, 1988.
29. Barksdale, R. D., S. Y. Itani, and T. E. Swor. Evaluation of Recycled Concrete, Open-Graded Aggregate, and Large Top Size Aggregate Bases. In *Transporation Research Record 1345,* TRB, National Research Council, Washington, D.C., 1992, pp. 92–100.
30. Goodwin, S., and M. W. Roshek. Recycling Project: Concrete Grinding Residue. In *Transportation Research Record 1345,* TRB, National Research Council, Washington, D.C., 1992, pp. 101–105.
31. Garrick, N. W., and K.-L. Chan. An Evaluation of Domestic Incinerator Ash for Use as Aggregate in Asphalt Concrete. Presented at the 71st Annual Meeting, TRB, Washington, D.C., 1992.
32. *Standard Specifications for Construction.* Mn/DOT, Minnesota, 1983, pp. 616–645.
33. Collins, R. J. Synthesis of Waste Utilization in Highway Construction. Presented at 67th Annual Meeting, TRB, National Research Council, Washington, D.C., 1988.
34. Buck, A. D. Recycled Concrete. In *Highway Research Report 430,* HRB, National Research Council, Washington, D.C., 1973, pp. 1–8.
35. Wolters, R. O., and R. H. Cassellius. *Evaluation of Road Waste Materials for Highway Uses.* Final Report, Inv. 638. LRRB, Minnesota, 1975, pp. 1–21.
36. Flynn, L. Recycling: Will Roads Become 'Linear Landfills'? *Roads and Bridges,* Vol. 30, No. 10, 1992, pp. 65–70.
37. Smith, L. L., and R. M. Ramer. Recycled Plastics for Highway Agencies. 71st Annual Meeting presentation, TRB, National Research Council, Washington, D.C., 1992.

38. P.C. Pavement Recycled. *Mid-West Contractor*, Vol. 139, No. 3663, 1976, pp. 16–22.
39. Ray, G. K. Recycle Old Concrete? *Highway and Heavy Construction*, Vol. 121, No. 1, 1978, pp. 30–31.
40. Barenberg, E. J., and M. R. Thompson. Design, Construction, and Performance of Lime, Flyash, and Slag Pavement. In *Transportation Research Record 839*, TRB, National Research Council, Washington, D.C., 1982, pp. 1–6.
41. Shuler, T. S., R. D. Pavlovich, and J. A. Epps. Field Performance of Rubber-Modified Asphalt Paving. In *Transportation Research Record 1034*, TRB, National Research Council, Washington, D.C., 1985, pp. 96–102.
42. Pavlovich, R. D., H. J. Lentz, and W. C. Ormsby. *Installation of Incinerator Residue as Base-Course Paving Material in Washington, D.C.*. Final Report, FHWA-RD-78-114. FHWA, U.S. Department of Transportation, 1977, pp. 1–21.
43. Halm, H. J. Econocrete. Presented at ASCE Spring Convention and Exhibit, 1977, pp. 1–29.
44. *Waste Tires in Sub-grade Road Beds*. MPCA, Minnesota, 1990, pp. 1–34.
45. Hoagberg, R. K., and V. Rajaram. *Minnesota Aggregate Resource Study*. Inv. 652. LRRB, Minnesota, 1980, pp. 66–68.
46. Gendreau, C. J. Recycling Concrete Pavement in North Dakota. In *Proc., Workshop in Pavement Rehabilitation*, Salt Lake City, FHWA, U.S. Department of Transportation, 1985, pp. 181–185.
47. Caylor, L. *Study to Determine Properties of Untreated RAP Materials for Use as a Base*. Final Report FHWA/GA/89/8814. FHWA, U.S. Department of Transportation, 1989, pp. 1–25.
48. Rubin, R. A., and J. L. Molina. In Too Deep. *Civil Engineering*, Vol. 62, No. 12, 1992, pp. 67–69.
49. Flynn, L. Jury Remains out on Asphalt-Rubber Use. *Roads and Bridges*, Vol. 30, No. 12, 1992, pp. 42–50.
50. *Standard Specifications of Construction*, Section 2361.2. Mu/DOT, St. Paul, 1983, pp. 262–263.
51. Radian Corp. Proposal for Clean Coal By-Product Utilization in Roadways, Embankments and Backfills, 1990–1991 Construction. RFP No. 3176-5, Vol. 1, 1990, pp. 1–7.
52. O'Neil, B. B., J. B. Gordon, D. R. Ehrich, and P. L. Bruner. *Minnesota Environmental Law*. Federal Publications Inc., 1992, pp. 530–545.
53. *Synthetic Precipitation Leach Test for Soils*. SW-846. Environmental Protection Agency, 1985.
54. Turgeon, C. M. *The Use of Asphalt-Rubber Products in Minnesota*. No. 89-06. National Seminar on Asphalt-Rubber, Kansas City, Mo., 1989.
55. Skok, E. L., and M. S. Kersten. *NCHRP 10-9: Bituminous Pavement Seal Coat Programming Procedures and Guidelines*. TRB, National Research Council, Washington, D.C., 1973.
56. Han, C., and A. Johnson. *Waste Products in Highway Construction*. Minnesota Local Road Research Board, 1993.

The opinions, findings, and conclusions expressed in this paper are those of the authors and not necessarily those of the Transportation Research Board.

Use of Industrial By-Products in Economical Standard Low-Volume Road Pavements

László Gáspár and László Gáspár, Jr., *Institute for Transport Sciences Limited, Hungary*

In Hungary, 50 percent of the urban and about 10 percent of the local rural roads are paved. The extremely limited amount of financial means available makes decreased costs for these low-volume pavements necessary. According to investigations by the Institute for Transport Sciences Ltd (KTI Rt) in Budapest, this goal can be attained by (*a*) utilization of relevant experience, (*b*) maximum possible use of industrial by-products and local materials with the appropriate techniques, (*c*) realistic design of pavements, and (*d*) staged construction of pavements. Use of various soil stabilization types, chemical treatments, and various wastes is discussed. Standard pavements and typical pavement cross sections are described.

In the 1950s and 1960s, based on the proposal of the forerunner of the present KTI Institute, Budapest, the base courses of about 150 low-volume roads were constructed using soil stabilization instead of the previously generally applied macadam base course. The slightly cohesive soils (silt and loess) were stabilized in situ using a cement binder. At that time, low-cost cutback binders were economical for the stabilization of fine sands. If nearby granular materials such as rock quarry wastes, silty gravel and sand, and slag were economically available, mechanical stabilization was preferred. The long-term behavior of these sections, some of which were considered experimental, was systematically monitored. The common experience gained from these sections was evaluated in 1987. These stabilization methods usually proved successful.

Early Experience with Pavements in Hungary

Cement Stabilization

A relatively thin asphalt surfacing on a cement-stabilized base course could endure a light traffic load for several decades. Major maintenance was necessary only if some construction fault, such as failure to apply cement, wetting of the subgrade (drainage deficiency), and construction in late autumn, occurred.

During the past 25 years, cement soil stabilization has become a widespread roadbuilding technique in Hungary. If local soils are used, the soil is generally milled on site; otherwise mixtures made in continuous-mixing plants are compacted using pneumatic-tire rollers.

Laboratory tests for the determination of the amount of cement needed have been developed and are included in standard specifications. The amount of cement used is important from technical and economical standpoints: a high cement content increases the strength unfavorably and is also uneconomical, whereas a low cement content does not provide the required bearing capacity and frost resistance.

The required cement rate is from 6 to 8 percent. Fine sand without silt, however, requires 10 to 12 percent cement content because of its high porosity. The quantity of cement can be reduced to 6 to 7 percent by mixing 15 to 20 percent silt with the material. The base courses of several low-volume streets in Budapest were constructed using cement-stabilized local sand.

Other Stabilization Techniques

Use of fine sand stabilized with cutback bitumen has proven successful for the base course of low-volume roads and was also economical because of the low bitumen prices. Currently, however, it cannot be considered a suggested technique.

As shown in Figure 1, a considerable part of Hungary is covered by silt (loess) and fine sand, which can be stabilized using various binders. Granular materials appropriate for mechanical stabilization are found only in a few areas of Hungary. If this technique is used properly, the results are favorable. In particular, wastes from rock quarries and gravel pits can be utilized advantageously for mechanical stabilization. Several rock wastes meet the grading requirements of mechanical stabilization. Other suitable materials are crushed blast furnace slag, burnt coal mine waste (red slag), coal and waste incinerator slag, and crushed debris from building demolition (*1*).

A paper by the author dealing in part with the location of quarry wastes and secondary materials suitable for road construction provides detailed information on the major sources of these materials. Favorable results have also been obtained from the rehabilitation, repair, and gradual development of unsurfaced roads using mainly mechanical stabilization (*2*).

Several experimental sections were constructed using slag from the Budapest Waste Incinerator Plant. On the basis of the results, technical guidelines on slag utilization were developed (*3*).

Lime stabilization was economically used where the low-volume road was on an acidiferous clay subgrade (for example, on a flood-prevention dam). The clay was not excavated, only loosened; the lime, either powdered slaked lime or a hydrated lime by-product, was admixed and, if necessary, wetted. The compaction was done by pneumatic-tire rollers or suitably guided loaded trucks.

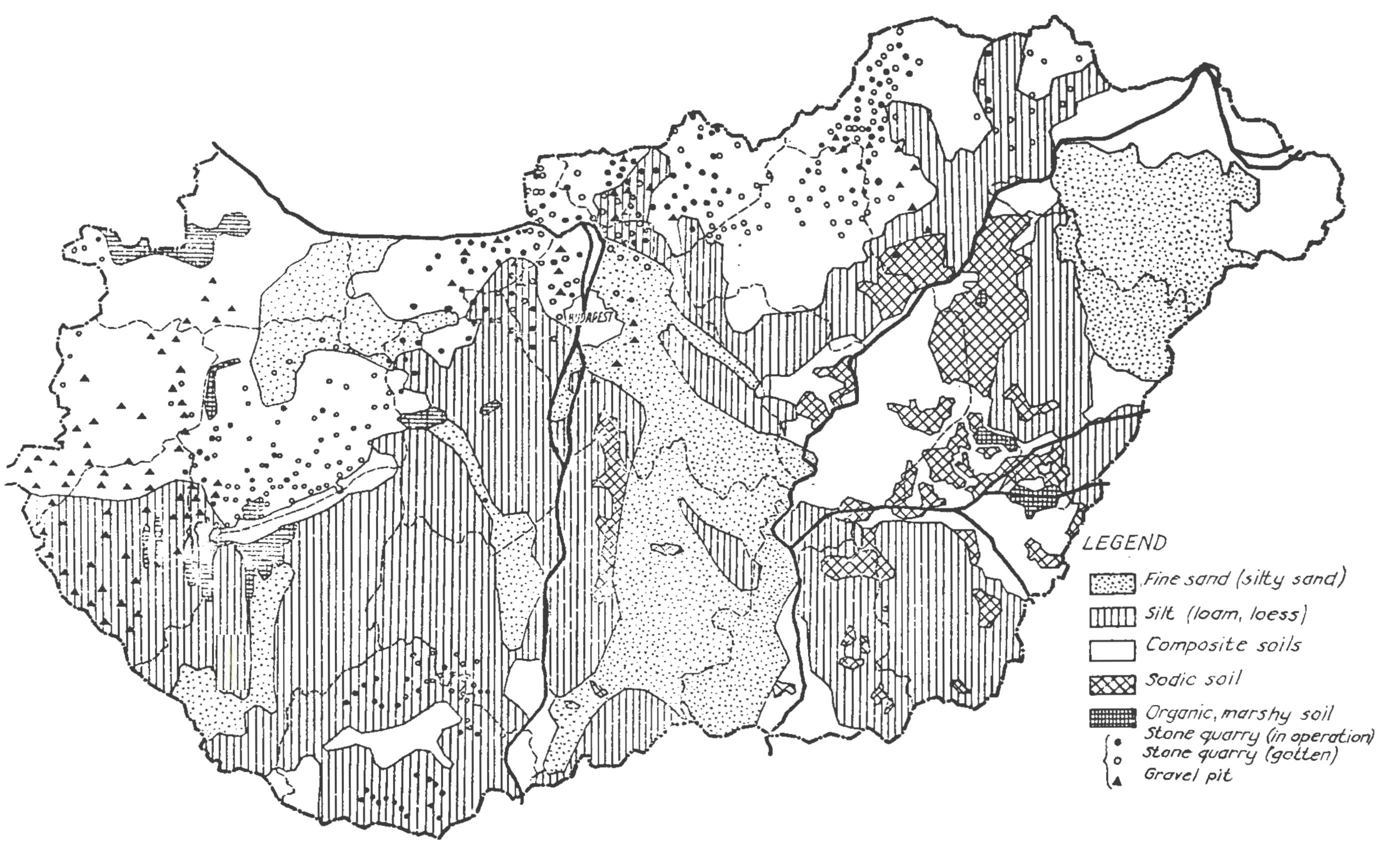


FIGURE 1 Schematic soil map of Hungary.

To determine the required lime rate, cement stabilization tests were used. Strengths at 28 and 42 days were considered critical because of the slower hardening.

Favorable results were obtained with the use of intermediate geotextiles when final or temporary pavement was constructed on a soft organic subgrade. When dirt roads were developed, surface and base courses were successfully replaced with geotextile impregnated with bitumen.

Fly-Ash-Bound Base Courses

Fly ash from thermal power plants proved an economical and appropriate substitute for cement binder in Hungary (based on French experience with this technique). The first three experimental sections were constructed in 1975. Favorable results caused development of the first technical directives, which have promoted the use of this technique. In Hungary, 5 million m^2 of fly-ash-stabilized base course have been constructed.

The majority of Hungarian power plants produce nonhydraulic fly ash that can be activated by the addition of lime. Instead of expensive lime, hydrated lime by-products and carbide caustic sludge can be used. Unused fly ash is transported to slime deposit areas where it settles down and dries. Wet fly ash activated by lime is also utilized as a binder.

When coal is burned with limestone waste, hydraulic fly ash with free CaO content of at least 18 percent is produced that becomes cement-like when mixed with water. This material can also be utilized for activating nonhydraulic fly ash. The fly-ash-bound mixtures bind slowly. This binding takes place in storage and makes for a longer compaction period for the mixture. The optimum binder rate is determined by laboratory tests that are the same as those for cement stabilization. The 60-day strength, however, is critical. The swelling of hydraulic fly ash during binding is compensated for by mixing it in an appropriate ratio with fly ash containing a high silica content. In a Hungarian power plant, various coal types of known composition were burned in a predetermined ratio to produce a hydraulic fly ash with the desired properties. To arrive at these ratios, Hungarian experts developed a new procedure to determine the equilibrium of fly ash–lime systems. A special device was established for the quick measurement of pozzolanic activity and active CaO content (*4*).

The construction cost of fly-ash base courses was 30 to 35 percent less than that of traditional cement variants when the hauling distance of the fly-ash and lime by-product was less than 60 km.

Technical directives for fly-ash base courses are being developed. Voids in low-quality crushed stone and blast furnace slag bases were filled successfully using cement or fly-ash mortar. This mortar ensured a waterproof base course.

Chemical Soil Treatment

In the 1970s, the local cohesive subgrades of several low-volume roads were treated with Reynolds Road Packer 235 (RRP-235) in regions of Hungary lacking granular materials. According to the manufacturer, on the basis of grading, condition limits, and compaction (Proctor) data of soils, the optimum chemical application rate as a function of soil particles smaller than 0.06 mm was determined to be 3 to 7 kg/100 m^2.

The upper 5-cm layer of leveled soil was loosened and the RRP-235, diluted with water, was sprayed in several applications. During the next 2 weeks, 20 to 30 mm of natural precipitation or sprinkled water made the chemical reach a few decimeters into the soil. Compaction was delayed until the moisture content of the treated soil approached the optimum compaction (Proctor) value. Too much rain sometimes made compaction difficult. One of the roads was treated in September 1970, but the compaction was carried out the following summer because of the rainy autumn and winter weather.

Chemische Bodenverbesserung (CBV) has been used in Hungary since 1977. The initial treatments proved successful; hence, laboratory tests were conducted using various soil types. These tests included measurement of the condition limits and the compaction dates of the untreated and the treated soils.

To obtain data on the waterproofness and bearing capacity of various soil types, the water absorption, swelling, and California bearing ratio (CBR) value of cylinders of various soil types subjected to 4-day capillary water action were measured. For some samples, the CBR value of the treated soils was more than 20 percent higher, and after the 4-day water action it was 5 percent lower than that of the untreated soils. For other samples, however, no improvement was observed. Based on these findings, the following conditions for the treatability of soils using chemicals were determined:

- Plasticity considerably decreases even under the influence of a few chemicals,
- The maximum dry (Proctor) density and the optimum compaction moisture content increase,
- The CBR value of the treated samples subjected to capillary water action is 15 to 20 percent higher than that of untreated soil, and
- The chemical soil mixture is acidic (pH less than 6).

It is important to treat only suitable soils using the appropriate procedures (*5*).

In the 1980s, more low-volume roads were constructed using chemical treatment. RRP-235 became available on the Hungarian market in the 1970s, and some test sections were constructed using it. Appropriate chemical soil treatment could result in considerable savings in areas having cohesive soils without granular materials.

Flexible Pavement Structural Design

In the early 1970s, a simple procedure was developed in Hungary using results of the AASHO Road Test and those of the Asphalt Institute for the structural design of flexible pavements (6). Pavements with various bearing capacities were compared on the basis of their H_e equivalent centimeter (ecm) thickness:

$$H_e = \Sigma\ e_i h_i$$

where e_i is the equivalency factor of each layer proportional to its load distribution capability, and h_i is the geometric thickness of each layer in centimeters.

The various pavement structural layer types were designated by letter symbols. The symbols for widespread stabilized base courses and their *e*-factors are as follows:

1. CK_t and PK_t, $e = 1.3$: cement and fly-ash-stabilized gravel (granular material), mixed in plant;
2. CK_h and PK_h, $e = 1.1$: the same layers as in 1, mixed in place;
3. CT_h and PT_h, $e = 0.8$: cement and fly-ash-stabilized soil, mixed in place; and
4. *M50*, $e = 0.8$: mechanically stabilized (D_{max} = max 50 mm).

Some asphalt pavement layers and surface dressings are as follows:

1. AB, KAB, HAB, EA, JU-20, $e = 2.2$: asphalt concrete, gravel asphalt concrete, cold asphalt concrete, emulsion asphalt, asphalt base course (partly crushed aggregate);
2. It-3, It-5, $e = 1.0$: penetration asphalt macadam layer; and
3. Fb_1 and Fb_2, $e = 0$: single and double surface dressing.

The bearing capacity of pavements has been characterized using the Benkelman beam since 1954. Later, Lacroix deflectographs and recently KUAB falling weight deflectographs have also been utilized.

Experimental Sections Under Controlled Traffic Load

In 1974, an experimental section was built on a fine sand subgrade as a part of a common program with the former German Democratic Republic to economically construct pavement for low-volume roads. The length of each type of pavement was 45 m, with the total length of the section being 1.7 km. One lane of the 6-m-wide roadway had surface dressing, whereas the other lane had an asphalt concrete course 2.5 cm thick. Underneath these surfaces were 35 different base course variants (for example, various cement stabilizations, bituminous base courses, and mechanical stabilizations). Periodically, loaded trucks passed over the experimental road. After each 2,500 passes, control measurements were made. The bearing capacity was characterized by deflection tests.

Based on evaluation of the results, the following observations were made:

- The bearing capacity of the fine sand subgrade (20 percent of the area of Hungary) is higher than had been supposed earlier: a minimum of 20 percent instead of 13 percent, and
- The thickness of the pavement can be considerably less than H_e = 20 ecm as specified in the Hungarian design guidelines (6).

The latter statement can be proved, for example, by the performance of a 12-km-long section of a main road constructed in 1971. A 65-cm layer of fine sand was placed on the cohesive clay subgrade. The upper 15 cm of this layer were stabilized by cement in situ. An asphalt pavement 11 cm thick was placed on top of the cement. According to the design guidelines (6), a much thicker pavement would have been necessary. Nevertheless, the road section still endures heavy traffic without major defects.

Standard Pavements

Local Soils, Bearing Capacity, and Base Courses

Local soil types have an important role in the design of pavements (subgrade bearing capacity) and in the design of base courses (stabilized soils). Figure 1 shows the distribution in Hungary of various soil types (such as silt, loess, fine sand, and clay) as well as sodic soil and organic and marshy soil.

For the design of low-volume roads, the following bearing capacity values can be used for the three main soil types:

- Clay, CBR = 5 percent;

- Silt (loess), CBR = 7 percent; and
- Fine sand, CBR = 15 percent.

It is important that the subgrade be constructed according to the relevant specifications and be well drained. Where this criteria cannot be met, the necessary subgrade bearing capacity should be attained using one of the following methods:

- Use of a granular protection layer,
- Lime treatment of wetted clay with an acidic chemical reaction, and
- Use of geotextile (as discussed in the section Other Stabilization Techniques).

Protection against spring thaw damage is needed according to the relevant specifications. In areas poor in granular materials, cement or fly-ash-stabilized base courses should be constructed using the local silt or fine sand and a mixed-in-place technique (ST_h or PT_h).

Industrial By-Products and Application Areas

Stone Quarry and Gravel Pit Wastes

Currently about 100 million tons of stone quarry and gravel pit wastes is available in Hungary. Yearly, about 1.6 million m^3 of gravel pit wastes and about 3.1 million tons of stone quarry wastes are produced. Figure 1 shows the regional distribution of the major stone quarries and gravel pits. Waste material from these can be utilized in the construction of MSO (M20) as well as CK and PK base courses. The utilization of these wastes is justified because of the savings of domestic rock resources as well as for economic and environmental reasons.

In addition to the big stone quarries and gravel pits, several hundred small pits are operated by agricultural firms, local councils, and forestry offices, among others. Waste material from these sources can often be obtained within a short hauling distance.

Other Granular Industrial By-Products

Crushed blast furnace slag can be substituted for natural granular aggregates in several areas of Hungary. In Ózd, crude iron is extracted from the old blast furnace slag waste site. The resulting crushed blast furnace slag should be utilized because repeated storage incurs significant costs. In Dunaújváros, not only crushed blast furnace slag but also crushed Siemens-Martin slag can be obtained, which can be used for surface dressings.

Slag from waste sites can be used for the construction of high-quality base courses if the uniform mixing of coarse and fine grains is ensured. The quantity of coal mine wastes exceeds 80,000 tons in Hungary. About half of it is red slag that burns as a result of spontaneous ignition. It can be used for the construction of high-quality mechanical stabilized bases. About 100,000 tons of waste incinerator slag produced in the Budapest Waste Incinerator Plant can be utilized for good mechanical stabilization because of its hydraulic fly ash content, which hardens slowly (*3*).

Mechanical stabilization can also be accomplished through use of boiler and grate slag, roof tile and brick fragments, as well as the debris from demolished buildings, if they meet the required specifications.

Fly Ash and Lime By-Products

Hungarian thermal power plants produce 5 million tons of fly ash a year. The following fly-ash-bound soils and gravels (granular materials) are found in the recommended pavement structures: PT_h, PT_t, PK_h, and PK_t (mixed in place or mixed in plant).

Design Parameters

In the gradual development of dirt roads, the pavement should usually be designed for a light traffic load, or Category A traffic. In this category, there should be several subcategories. The planned life cycle of the first step can be 5 years. Relatively light commercial vehicles with maximum weight of 60 to 80 kN run on these secondary roads. Regular bus traffic is usually IKARUSZ 250 buses with a total weight of 110 kN. During the first years, the actual load from these vehicles is generally under the maximum permitted value. Some two-thirds of the load falls on the rear dual-wheel axle. Thus, the standard axle load can be taken as 80 kN, supposing a 10-kN effective load.

When converting into F 100-kN unit axles, factors of 0.08, 0.16, and 0.30 are utilized for 60-, 70-, and 80-kN axles. Accordingly, the standard pavements should be designed according to the following traffic subcategories:

- A_1: 1,000 F100: no regular bus service, 4 to 5 commercial vehicles per day with a 60- to 70-kN load,
- A_2: 2,000 F100: twice the value of an A_1 load,
- A_3: 5,000 F100: 4 bus runs per day in addition to an A_2 load, and
- A_4: 10,000 F100: 14 bus runs and about 15 commercial vehicles per day.

Commercial vehicles under 60 kN (such as trailers towed by tractors) or passenger cars should not be considered in the design.

The width of the pavement is as follows:

- A_1 and A_2 traffic subcategories: 3.5 m and 4.0 m and
- A_3 and A_4 traffic subcategories: 6.0 m.

For the A_1 and A_2 traffic subcategories, the vehicle conversion factor is 1.0. That is, the loads of both traffic lanes should be taken into account. For the A_3 and A_5 subcategories, a factor of 0.7 is used when 5,000 and 10,000 F100 traffic loads are calculated.

The design bearing capacities of clay, silt, and sand-subgrades are CBR = 5, 7, and 15 percent, respectively.

Base and Wearing Course Materials

The base courses of the four standard pavements (T_1, T_2, T_3, and T_4) are as follows:

- T_1 M50 mechanical stabilization (e = 0.8): This base type can be constructed using a simple apparatus. It proves to be economical only if transportation costs are not too high.
- T_2 CT_b or PT_b cement or fly-ash-stabilized silt (loess) or sand mixed in place (e = 0.8): Use of these bases is economical if no local granular material is available. Site mixing can be considered efficient. The maximum layer thickness that can be mixed in a run is 20 cm. The mixed-in-plant CT_t and PT_t variants with e = 1.0 are rarely applied.
- T_3 CK_b or PK_b cement or fly-ash-stabilized gravel (granular material), mixed in place (e = 1.1): granular material in the vicinity increases the economy.
- T_4 CK_t or PK_t cement or fly-ash-stabilized gravel (granular material), mixed in plant (e = 1.3).

If suitable chemicals can be economically obtained, treatment of the cohesive soils with an acidic chemical reaction can also be used in areas with fewer granular materials.

Standard Pavement Variants

The following are the equivalent thicknesses of the 12 standard pavement variants, subgrade bearing capacity groups I through III and traffic subcategories A_1 through A_4:

F100 Subgrade CBR (%)	*H_e (ecm)* A_1	A_2	A_3	A_4
Group I: 5	21	24	28	32
Group II: 7	18	21	25	28
Group III: 15	14	16	18	21

The favorable base courses of various standard pavements can be selected as a function of the local conditions as follows:

- Group T_1, e = 0.8, M50 mechanical stabilization;
- Group T_2, e = 0.8, CT_b or PT_b;
- Group T_3, e = 1.1, CK_b or PK_b; and
- Group T_4, e = 1.3, CK_t or PK_t.

Figures 2 and 3 present the standard pavements T_1, T_2, T_3, and T_4.

The CT_t and PT_t variants with e = 1.0 equivalency factors are rarely applied, and no special standard pavement group was designed for them. However, if preferred, the thicknesses of group T_3 (e = 1.1) could be modified as follows:

- A_1 I, II, III: 21, 18, and 14 cm, respectively;
- A_2 I, II, III: 21, 21, and 16 cm, respectively;
- A_3 I, II, III: 22, 22, and 18 cm, respectively; and
- A_4 I, II, III: 21, 22, and 15 cm, respectively.

Figure 4 illustrates the typical cross sections. Pavements in groups T_1 through T_3 are considered flexible, whereas the

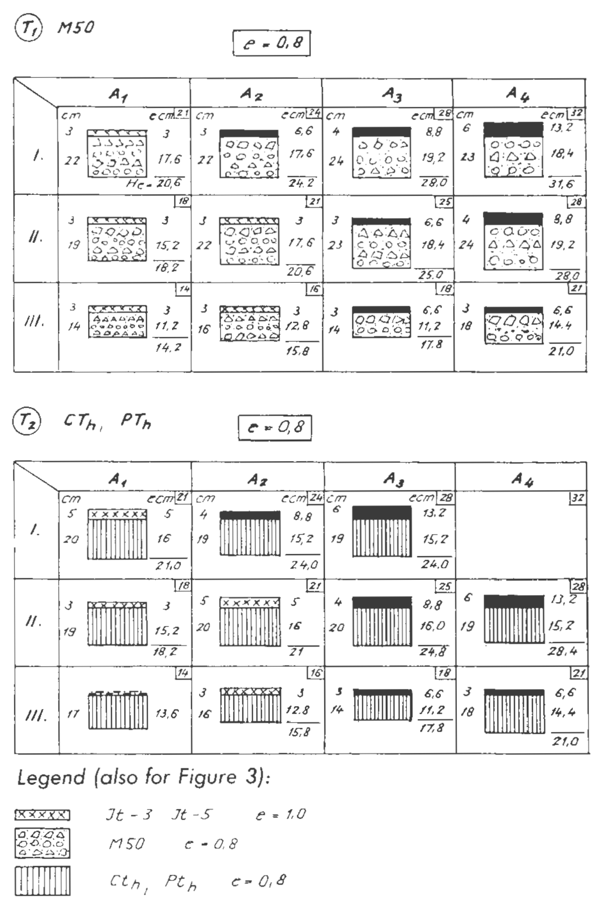


FIGURE 2 Standard pavements T_1 and T_2.

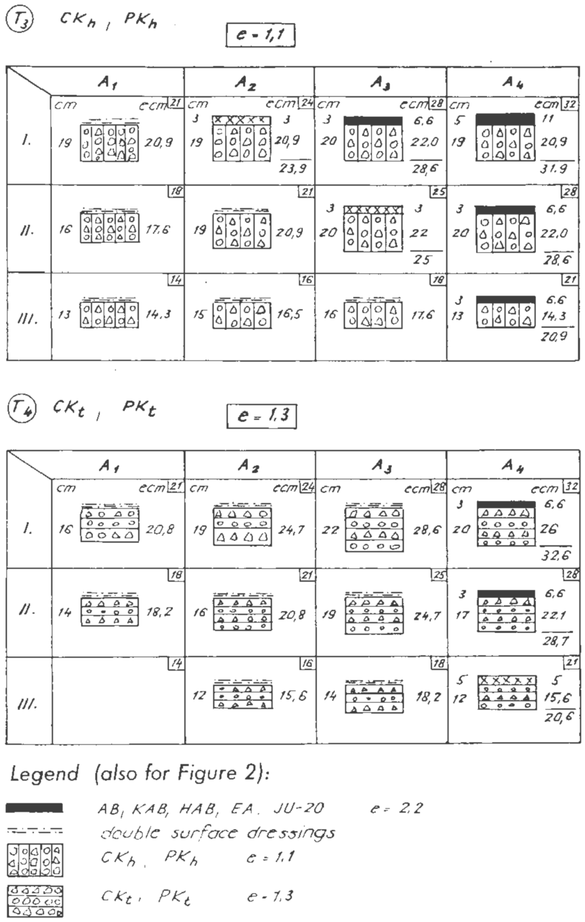


FIGURE 3 **Standard pavements T_3 and T_4.**

ones in group T_4 are semirigid variants. Accordingly, the allowable deflection values in 0.01 mm are as follows:

Group	A_1	A_2	A_3	A_4
T_1–T_3	300	270	230	190
T_4	250	220	180	150

Economic Analysis

When the optimum pavement type is being selected, it is expedient to investigate at least three favorable variants. Economy is considered the first criterion in ranking. Information about construction costs is enhanced by county (regional) costs in HUF/m^2/ecm.

Another aspect is the machinery needed. Generally, short sections should be constructed using low standards because setting up a mixing plant would considerably increase construction costs. That is why the base courses of group T_1 made without mixing if low-cost granular material is available or the base courses of groups T_2 and T_3 produced by the mixed-in-place technique would be preferred.

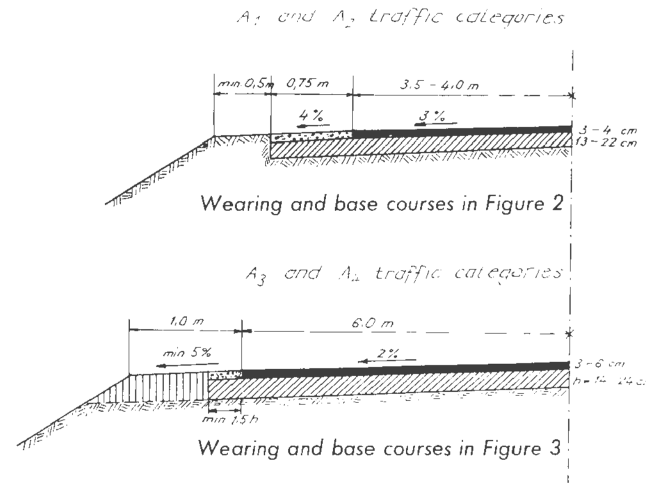


FIGURE 4 **Typical pavement cross sections.**

The equivalency costs of penetration asphalt macadam pavements are not favorable although their single machinery need can make the technique competitive, especially in the case of sealing of M50 base courses. The main advantages of HAB and EA cold asphalts are that they can be transported from a relatively long distance and can be stored for a long time.

The mixed-in-plant base courses (CK_t and PK_t or, rarely, CT_t and PT_t) and the hot-asphalt pavements (KAB, AB, JU + F_b) are preferred if a mixing plant operates close to the working site and the materials can be obtained economically from that plant.

It is expedient to use precoated chippings for surface dressings because the run would be very slow because of the low-volume traffic causing intensive raveling.

REFERENCES

1. Gáspár, L. L'utilization des sous-produits industriels dans la construction routière en Hongrie. *Revue Générale des Routes et Aérodromes*, Oct. 1982.
2. Gáspár, L. Restauration, amélioration, et développement progressif des voies en terre agricole en Hongrie. *Proc., 11th International Congress on Agricultural Engineering*, CIGR, Dublin, Ireland, 1989.
3. Gáspár, L. Verwendungsmöglichkeiten der Budapester Müllverbrennungsasche (-schlacke) in stadtischen Tiefbau. *Proc., Ninth Danube-European Conference on Soil Mechanics and Foundation Engineering*, Budapest, Hungary, Oct. 1990.
4. *The Use of Fly Ash in Road Construction.* Highway Transport Division, Ministry of Transport, Budapest, Hungary, 1986 (in Hungarian).
5. Gáspár, L. Essais du traitement chimique du sol en Hongrie. *Proc., 10th International Conference on Soil Mechanics and Foundation Engineering*, Stockholm, Sweden, June 1981.
6. *Design Guidelines for Flexible Pavement Structures.* Ministry of Traffic and Communication, 1971 (in Hungarian).

Field Performance of Tire Chips as Subgrade Insulation for Rural Roads

Dana N. Humphrey, *University of Maine*
Robert A. Eaton, *U.S. Army Engineer Cold Regions Research and Engineering Laboratory*

This paper describes a field trial that uses tire chips as an insulating layer to limit frost penetration beneath a gravel-surfaced road. Tire chips are an attractive alternative to conventional insulation boards because they have a high thermal resistivity and are durable, free draining, and low-cost. The primary goals were to determine the thickness of tire chips needed to provide effective insulation and the minimum thickness of overlying soil cover needed to produce a stable riding surface. The project is 230 m (750 ft) long and consists of five sections with either a 152-mm (6-in.) or a 305-mm (12-in.) tire chip layer overlain by 305 mm (12 in.), 457 mm (18 in.), or 610 mm (24 in.) of granular soil. In addition, there are three control sections. The project is instrumented with thermocouples, resistivity gauges, and groundwater monitoring wells. Based on an analysis of the first two winters in service, a 152-mm-thick tire chip layer overlain by 305 mm (12 in.) of gravel reduced the depth of frost penetration by 22 to 28 percent compared with an adjacent control section. Likewise, a 305-mm (12-in.) tire chip layer overlain by either 457 mm (18 in.) or 610 mm (24 in.) of gravel resulted in a 15 to 37 percent reduction. Furthermore, the two sections with 305 mm (12 in.) of tire chips experienced a heave of between 10 mm (0.4 in.) and 40 mm (1.6 in.), whereas a nearby control section heaved 55 mm (2.3 in.) to 91 mm (3.6 in.).

Maintaining gravel-surfaced roads in northern climates during the spring thaw is a perennial problem. Ice lenses form in the underlying soil if the soil is frost susceptible and the groundwater table is near the road surface. As temperatures warm, these ice lenses melt, releasing significant quantities of excess water. Melting proceeds downward from the road surface. The water cannot drain down because the underlying soil is still frozen. Moreover, the ditches at the side of the road are often still filled with snow and ice, so the water cannot drain laterally through the granular surface course. The water is forced up to the road surface, which saturates and weakens the granular surface course. Traffic ruts the surface, which ruins the cross drainage and traps more water on the road surface. Regrettably, this is a common occurrence since the soil types found in northern climates, such as glacial tills, poorly sorted outwash sands and gravels, and marine and lacustrine clays, are often very frost susceptible. Moreover, most northern climates are relatively wet and produce a high groundwater table.

Three remedies to reduce the rutting of the road surface during the spring melt are (*a*) to increase the thickness of the granular surface course; (*b*) to improve the drainage to lower the groundwater table; or (*c*) to use insulation to prevent the underlying frost-susceptible soil from freezing. The first two remedies are the most common. However, the granular surface course must be thick to completely solve the problem. Roads with a 457-mm (18-in.) granular surface course have experienced significant rutting during the spring melt. Improving drainage by increasing the depth of the ditches along the side of the road is a low-cost solution, but local topography and the possibility that vehicles may slide into the ditch may make it impossible to provide ditches deep enough to solve the problem. The third remedy, insulation, is rarely used on rural roads because of the high cost of extruded polystyrene insulation boards.

This paper presents results of a field trial using a new insulating material, tire chips, to limit the depth of frost penetration beneath a gravel-surfaced road. Tire chips are waste tires that have been cut into 51-mm (2-in.) pieces or smaller. Tire chips are an effective insulator because rubber has a much lower thermal conductivity than soil does (*1,2*). Moreover, tire chips have a very high permeability (*3,4*) and provide excellent drainage to remove excess water from the road substructure.

Using tire chips as an insulating layer has important implications for waste disposal since every cubic meter of tire chip fill contains 100 tires (75 tires/yd^3). A kilometer of two-lane road underlain by a 305-mm (12-in.) layer of tire chips would require about 200,000 tires (300,000 tires/mile). Tire chips are produced by portable shredding machines, so it would be possible to produce tire chips at local solid waste disposal facilities and leave behind a pile of road building material instead of a solid waste disposal problem. In addition, this application could use some of the more than 2 billion scrap tires that have been discarded in huge open piles across the United States (*5*).

The field trial was designed to answer three primary concerns: (*a*) the thickness of tire chips needed to provide adequate insulation; (*b*) the thickness of granular soil cover needed over the tire chips to provide a stable riding surface; and (*c*) the effect of tire chips on groundwater quality. Several thicknesses of the tire chip layer and overlying granular soil cover were tested. An extensive instrumentation and monitoring program was established that included using thermocouples and resistivity gauges to monitor thermal behavior, measuring surface deflections of the road with a heavy weight deflectometer; and monitoring groundwater quality in several monitoring wells. This paper will concentrate on the thermal behavior of the field trial using data from the first two winters. A companion paper focuses on the support characteristics of the tire chips (6). The groundwater quality study is ongoing, but laboratory tests suggest that there is little likelihood that tire chips will release contaminants in sufficient quantities to be of concern (*7–9*) as long as chips are placed above the groundwater table. Other field studies are under way at the University of Maine to gather more data on the effect of tire chips on water quality when they are placed below the groundwater table. Construction of the test section and data from the first winter were discussed in detail in previous papers (*10,11*).

Site Description

The test site is located on Dingley Road in the town of Richmond, Maine. The dead-end, gravel-surfaced road serves 29 residences and two farms. The predominant traffic is cars, light trucks, and school buses. However, one day a month, 10 to 40 fully loaded double- and triple-axle dump trucks haul sewage sludge to farms located at the end of the road. Residents report that the road surface becomes severely rutted during the annual spring melt.

The road follows the northeast shoulder of a broad, flat ridge that trends northwest to southeast. During the summer and fall, no standing water or wet areas are evident near the test site. However, during the spring melt, the generally flat topography leads to poor drainage and areas of standing water.

In most areas, the existing road was surfaced with more than 457 mm (18 in.) of clean sandy gravel and gravely sand. The underlying native soils ranged from gray silty clay to gray-brown silty gravely sand. These soil types are highly frost susceptible. Probes were conducted with a 127-mm (5-in.) diameter power auger. Refusal occurred at depths ranging from 2.7 to 5.6 m (9 to 18 ft). The general geology of the area suggests that refusal was glacial till with boulders or bedrock. The water table in the summer and fall is 1 to 3 m (3.3 to 9.8 ft) below the ground surface.

Test Site Configuration

The test site is 290 m (950 ft) long and is broken up into five tire chip test sections, each with a length of 23 m (75 ft) or 46 m (150 ft). The tire chip test sections are designated Sections A through E. In addition, there is one 46-m (150-ft) long control section and two 30-m (100-ft) long transition sections. Two sections of the existing road are also used as controls. The location of the sections is shown in Figure 1. Two different thicknesses of tire chips, 152 and 305 mm (6 and 12 in.), were used to investigate the thickness required to provide adequate insulation, and three different thicknesses of granular soil, 305, 457, and 610 mm (12, 18, and 24 in.), were placed over the tire chips to investigate the thickness necessary to provide a stable riding surface. The granular soil cover includes a 102-mm (4-in.) thick

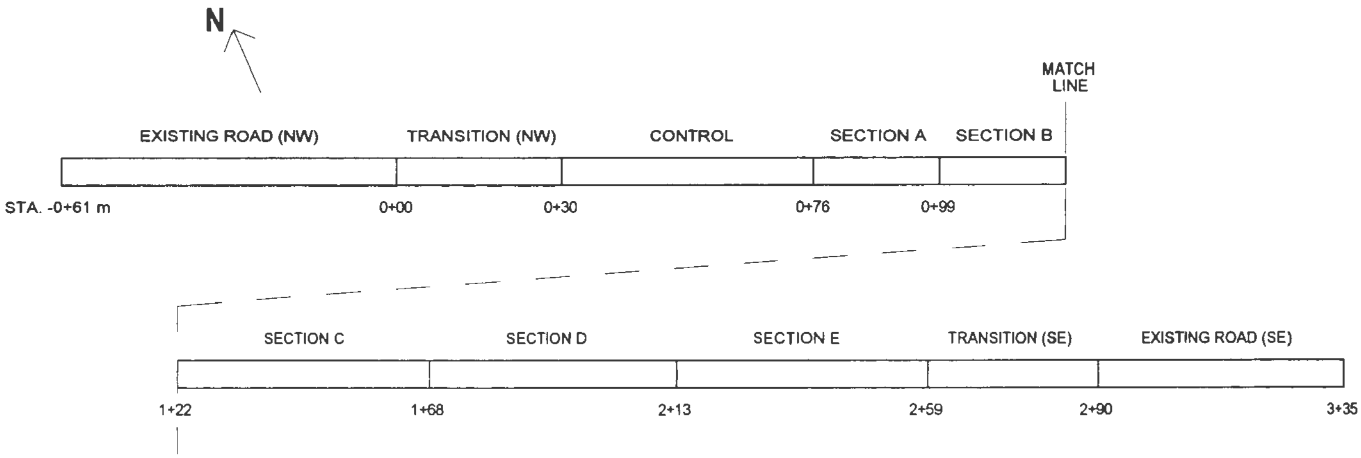


FIGURE 1 Plan view of test site.

granular surface course with a maximum particle size of 25 mm (1 in.). This provides a smooth, low-maintenance riding surface. A typical cross section is shown in Figure 2. Before the tire chips were placed, the existing road surface was excavated to between 152 and 457 mm (6 and 18 in.) to keep the final grade of the road surface from being too far above the surrounding terrain. The bottom of the excavation was sloped toward the ditch to enhance drainage. A minimum of 610 mm (2 ft) of gravel cover was maintained between the edge of the tire chip course and the 3:1 side slopes.

The configuration of the test sections is summarized in Table 1. The thicknesses of the soil and tire chip layers in Sections A and B were identical. However, the tire chips in Section A were completely enveloped in a woven geotextile (Amoco 2000-2) to evaluate the need for a geotextile to act as a filter to minimize infiltration of the underlying and overlying soils into the tire chip layer.

Materials

The tire chips were uniformly graded and had a nominal maximum size of 51 mm (2 in.). Almost all the tire chips were retained on the No. 4 (4.75-mm or 0.187-in.) U.S.-standard sieve. The chips were made from a mixture of steel and glass-belted tires and were irregular

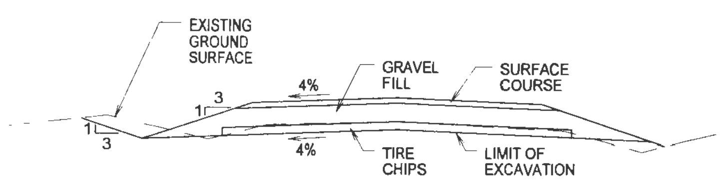


FIGURE 2 Typical cross section.

TABLE 1 Summary of Test Section Configuration

Section	Depth of excavation (mm)	Thickness of layer (mm)			
		Tire chips	Common borrow	Gravel fill	Surface course
Control	----	----	----	203	102
A	152	152	----	203	102
B	152	152	----	203	102
C	152	152	----	356	102
D	305	305	----	356	102
E	457	305	305	203	102

25.4 mm = 1 in.

in shape; many had steel belts protruding from the cut edge. They were donated by Pine State Recycling of Nobleboro, Maine. Approximately 20,000 tires were used in this small project. There is great potential for this application to use large quantities of waste tires.

The gravel fill used over the tire chips was a well-graded mixture of sand and gravel, with less than 5 percent passing the No. 200 (0.075 mm; 0.00295 in.) U.S.-standard sieve and a maximum particle size of 152 mm (6 in.). The surface course was obtained from the same source as the gravel fill; however, it had a maximum particle size of 25 mm (1 in.) and about 7 percent passing the No. 200 sieve. Common borrow was used as part of the soil cover over the tire chips in Section E to reduce the quantity of imported granular fill. The common borrow was salvaged from granular soil excavated from the existing road surface. It was a gravely sand with about 3 percent passing the No. 200 sieve.

Construction

The test section was constructed from August 24 through September 2, 1992. The first step was to excavate the northwestbound lane of the existing road down to the desired starting grade. This was done with a wheel-mounted hydraulic excavator. Excavated soil was hauled away with 11-m^3 (14-yd^3) capacity dual-rear-axle dump trucks. Some of the soil was stockpiled near the site for later use as common borrow; the remainder was disposed of off-site. The grade was smoothed with a small bulldozer and given the specified 4 percent slope toward the ditch. The exposed grade was then compacted with four passes of a vibratory smooth drum roller with a static weight of 9 metric tons (10 U.S. short tons).

The tire chips were hauled to the site in a 12-m (40-ft) long self-unloading semitrailer. About 20 metric tons (22 U.S. short tons) was hauled in a single load. The tire chips were unloaded directly on the prepared subgrade and then spread to the desired thickness with a small bulldozer. The bulldozer could easily achieve the specified grade within ±12 mm (0.5 in.).

The tire chips were compacted with six passes of a smooth drum vibratory roller with a static weight of 9 metric tons (10 U.S. short tons). From visual observation, the first pass caused a 305-mm (12-in.) thick layer of tire chips to be compacted by 10 to 25 mm (0.45 to 1.0 in.). Compaction on subsequent passes was too minor to be observed.

After the tire chips were placed, they were covered with the specified thickness of gravel fill or, in Section E, by common borrow followed by gravel fill. The specified thicknesses are summarized in Table 2. The gravel cover and common borrow were hauled to the site in 11-m^3 (14-yd^3) capacity dump trucks, spread in a 305-mm (12-in.) maximum thickness lift with a small bulldozer, and then compacted with six passes of a smooth drum vibratory roller with a static weight of 9 metric tons (10 U.S. short tons).

During construction, three in-place density tests were performed on the gravel fill, and one was performed on the common borrow. Compared with the results of modified Proctor compaction tests, the water contents were 2 to 3 percentage points dry of optimum and the percent compactions were 78 to 88 percent (*10,11*). The low water contents were undoubtedly a contributing factor in the low compacted densities. The difficulty of compacting granular soil placed on the compressible tire chips may also be important.

Finally, the 102-mm (4-in.) thick surface course was placed on the gravel fill. It was hauled, spread, and compacted in a manner similar to the gravel fill process except that only four passes were made with the roller. Final shaping was performed with a small road grader. The completed surface was treated with flake calcium chloride. The complete construction specifications for the project are given by Humphrey (*12*).

Monitoring Program

An extensive monitoring system was put in place to evaluate the thermal behavior, road surface support characteristics, and groundwater quality of the project. Installed instrumentation includes vertical strings of

TABLE 2 Depth to Groundwater Table

Well no.	Station (m)	Section	Depth from road surface to ground water table (m) 11/24/92	12/23/93
1	0+21.0	Transition (NW)	2.84	1.05
2	0+91.5	A	2.50	0.89
3	1+04.3	B	2.15	0.59
4	1+88.7	D	2.36	1.08
5	2+06.4	D	1.44	0.80
6	2+53.7	E	1.66	1.37

1 m = 3.28 ft

thermocouples installed at two locations in each of the five tire chip test sections, the control section, and a section of the existing road; resistivity gauges to monitor the location of the freezing front in each test section, the control section, and the existing road; six groundwater monitoring wells; and two frost-free bench marks. The thermocouples and resistivity gauges were connected to a system that allowed them to be read by telephone from the Cold Regions Research and Engineering Laboratory in Hanover, New Hampshire. The instrumentation plan and cross section are shown in Figure 3. Details of the monitoring program follow.

Thermocouples and Resistivity Gauges

Two vertical strings of thermocouples were installed in each test section, the control section, and the existing road. Each string consisted of twelve 20-gauge copper constantan thermocouples. The vertical spacing between thermocouples varied from 76 mm (3 in.) near the road surface to 305 mm (12 in.) at greater depths. The deepest thermocouple was typically about 2 m (6.5 ft) below the road surface. The installation in a typical section (Section C) is shown in Figure 4. To maintain the desired spacing, the thermocouples were mounted in a 25-mm (1-in.) diameter wooden dowel. The thermocouple strings were installed in a 127-mm (5-in.) diameter hole drilled with a trailer-mounted power auger. After the string was placed, the hole was backfilled with native soil tamped in place with a hand tamper.

The thermal resistivity gauges consisted of 25-mm (1-in.) diameter copper rings spaced 51 mm (2 in.) apart on an epoxy-filled core. The electrical resistivity of the soil between adjacent rings was measured to determine if the soil was thawed or frozen. The resistivity of frozen soil was much lower than that of thawed soil. This allowed the location of the freezing front to be monitored during the winter; the thawing front was also monitored during the spring. The resistivity gauges were 1.2 m (4 ft) long. The top of the gauge was typically even with the bottom of the tire chip layer. The installation technique was the same as that for the thermocouples. A typical installation is shown in Figure 4.

Groundwater Wells

Groundwater monitoring wells were installed at six locations so that water quality samples could be taken and the elevation of the groundwater table could be measured. The wells consisted of 51-mm (2-in.) diameter Sch. 40 polyvinyl chloride (PVC) pipe. A cap was glued to the bottom of the pipe and a hacksaw was used to cut slots in the cap and the bottom 0.6 m (2 ft) of the pipe. The pipe was placed in a 127-mm (5-in.) diameter hole drilled with a trailer-mounted power auger. The slotted lower portion was then surrounded with concrete sand. A 0.5-m (1.5-ft) thickness of bentonite balls was then placed to form an impermeable seal to prevent surface water from reaching the slotted tip. The remainder of the hole was backfilled with native soil.

One well was located next to the control section to provide background readings of water quality. The remaining five wells were located next to the tire chip test sections. Water quality samples will be taken on a quarterly basis and monitored for metals such as iron and manganese that could potentially leach from the tire chips. Results from groundwater quality monitoring are not yet available.

Heave Survey

The heave of the road surface was measured several times during the winter with a level survey. Two frost-

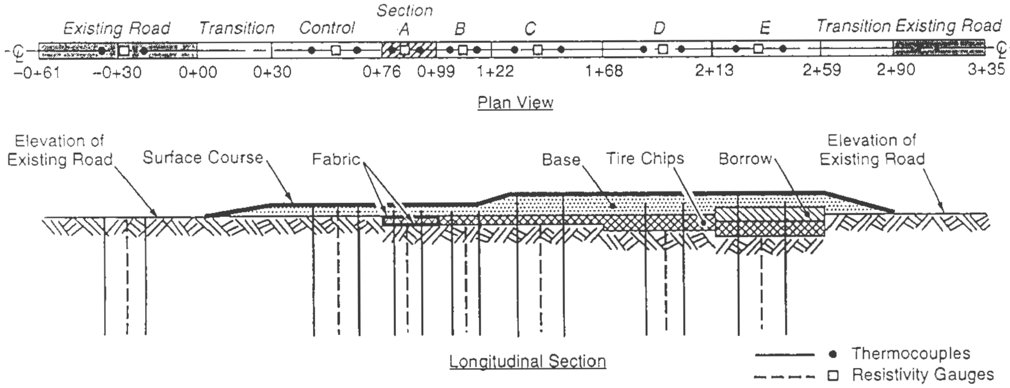


FIGURE 3 Location of instrumentation.

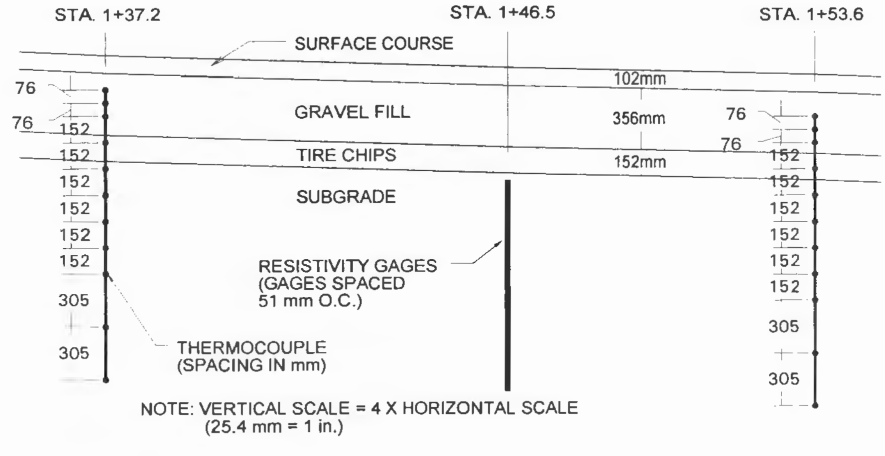


FIGURE 4 Cross section showing thermocouples and resistivity gauges in Section C.

free benchmarks were installed to provide stable reference points for this survey.

Performance

The test sections were in place for the winters of 1992 to 1993 and 1993 to 1994. The freezing index for 1992 to 1993 ws 626°C-days (1128°F-days); for 1993 to 1994, it was about 707°C-days (1273°F-days). Both winters were somewhat colder than the reported average for the area of 470°C-days (850°F-days) (*13,14*). The following summary concentrates mostly on the data from the 1993 to 1994 winter. A more complete presentation of data from the winter of 1992 to 1993 is given by Humphrey and Eaton (*10,11*).

Thermal Behavior

The maximum depth of frost penetration for the winters of 1992 to 1993 and 1993 to 1994 is summarized in Figure 5. The depth of frost penetration beneath the existing road and control sections ranged from 1170 mm (46 in.) to 1600 mm (63 in.). In contrast, in tire chip Sections A, B, D, and E, the depth of frost penetration ranged from 910 mm (36 in.) to 1021 mm (40 in.). The tire chips reduced the depth of frost penetration by between 22 and 28 percent compared with the control section. In Section C the depth of frost penetration was 1040 mm (41 in.) during the winter of 1992 to 1993 and 1205 mm (47 in.) during the winter of 1993 to 1994. This indicates that the thicknesses of soil cover and tire chips in Section C are not as effective in reducing the depth of frost penetration as the tire chips in the other sections. One reason that frost penetration depth was greater in Section C than in Sections A and B, which have 152 mm (6 in.) of tire chips, is that the effectiveness of insulation decreases as thickness of the overlying soil cover increases, as indicated by design charts presented by Berg and Johnson (*15*). In Sections C and D, which both have 457 mm (18 in.) of soil cover, increasing the thickness of the tire chip layer from 152 mm (6 in.) to 305 mm (12 in.) reduced the depth of frost penetration by 70 mm (3 in.) in the 1992 to 1993 season and 307 mm (12 in.) in the 1993 to 1994 season. In Section E, the frost penetrated 45 to 76 mm (2 to 3 in.) below the bottom of the tire chip layer.

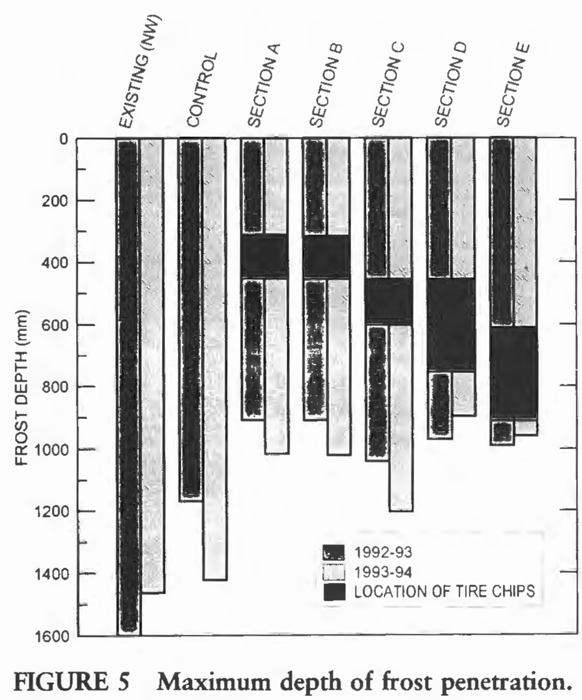


FIGURE 5 Maximum depth of frost penetration.

The depth of frost penetration compared with time for the 1993 to 1994 season shows even more clearly the effect of tire chips (Figure 6). In the existing road and control section, the frost penetrated to a depth of about 750 mm (30 in.) within about 1 week of the onset of the freezing season. After this, the frost continued to penetrate at an approximately constant rate for the remainder of the freezing season. In the tire chip sections, there was also a rapid initial penetration of the frost; however, after this, the rate of frost penetration was considerably lower than that beneath the existing road and control section. In Sections A and B, the rate of frost penetration decreased to almost zero after February 1, 1994. From February 1, 1994, through February 24, 1994, the frost penetrated an additional 18 mm (0.7 in.) in Section A. In contrast, the depth of additional frost penetration in the adjacent control section over the same period was 122 mm (4.8 in.). In Section D, there

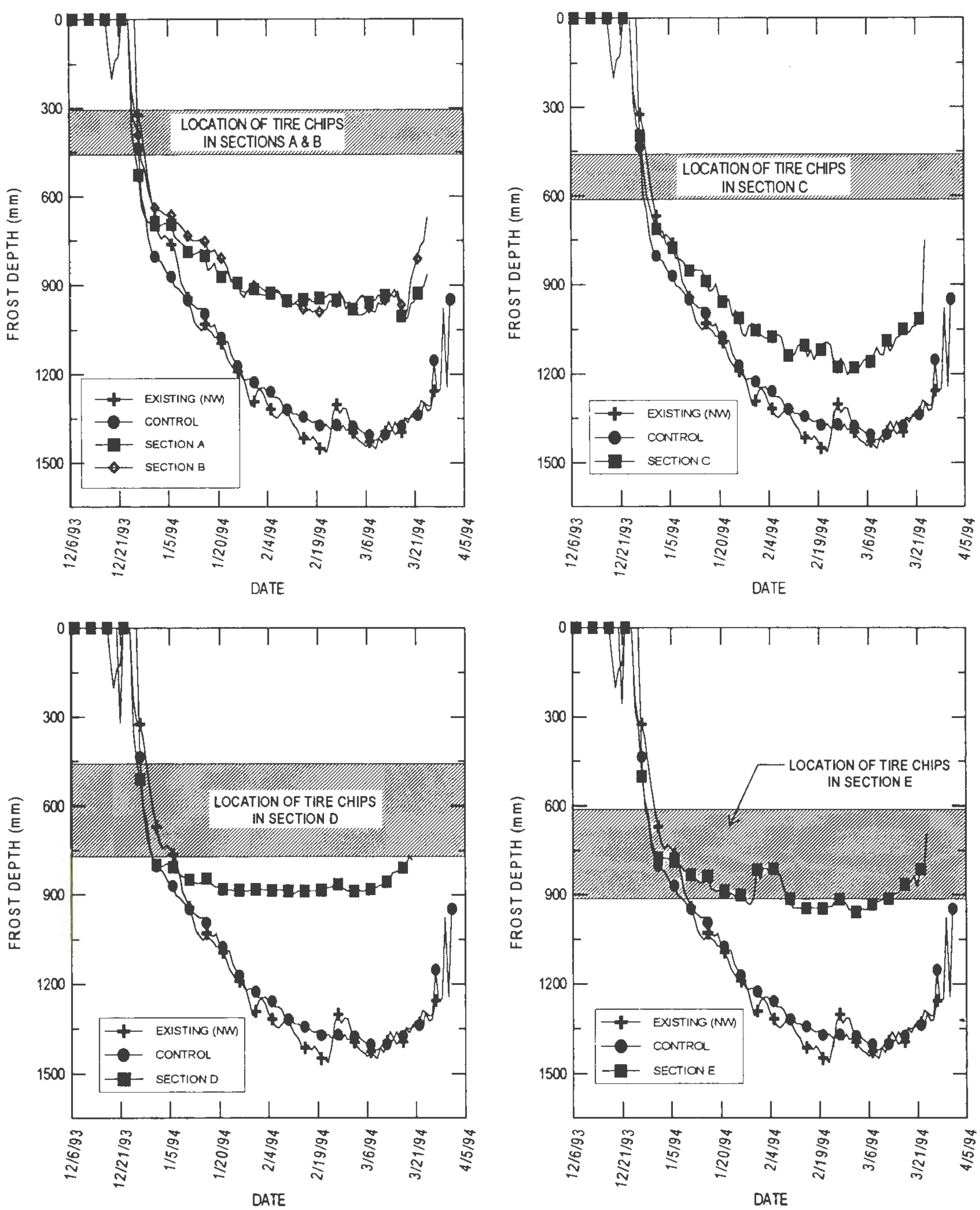


FIGURE 6 Depth of frost penetration versus date.

was little additional frost penetration after the frost penetrated the tire chip layer; in Section E, the frost remained within the lower portion of the tire chip layer for most of the freezing season, as shown in Figure 6. Similar results were found for the 1992 to 1993 season, as discussed by Humphrey and Eaton (*10,11*).

Comparisons of temperature and depth show the beneficial effects of tire chips. February 16, 1994, marked the end of a 45-day cold period in which the average daily temperature typically ranged between −5°C and −20°C (23°F to −4°F). The temperature beneath the existing road and control sections increased roughly linearly with depth, as shown in Figure 7. However, in the tire chip sections, there was a marked increase in temperature from the top to the bottom of the tire chip layer. The temperature increased 3°C to 4°C (5°F to 7°F) for the sections with 152 mm (6 in.) of tire chips and 8°C for sections with 305 mm (12 in.) of tire chips. This indicates that the tire chips have a higher thermal resistivity than that of the soil. Figure 7 also shows that in Sections D and E, the temperature of the soil immediately above the tire chip layer was 2°C to

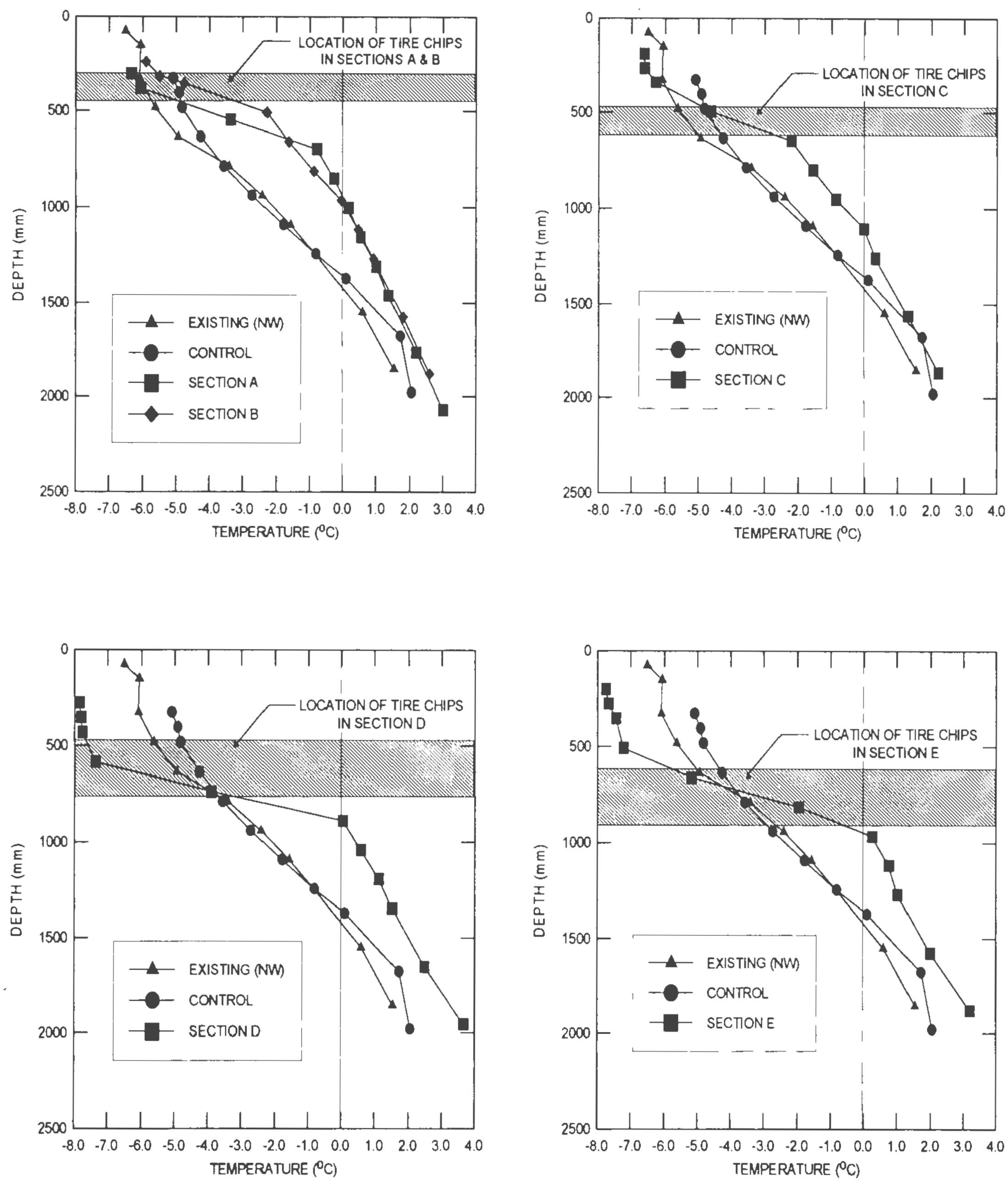


FIGURE 7 Temperature profile on February 16, 1994.

3°C (4°F to 5°F) colder than the same depth in the existing road and control section. This was because the higher thermal resistivity of the tire chips reduced upward flow of heat from the underlying warmer soil. Results for the 1992 to 1993 season were similar (*10,11*).

Groundwater Levels

Groundwater levels were measured in six monitoring wells. Readings taken near the beginning of the freezing season are shown in Table 2. For the readings before the 1993 to 1994 season, the groundwater table was 0.59 m (1.9 ft) to 1.37 m (4.5 ft) below the road surface. Comparison with Table 1 shows that the groundwater table is 0.04 m (0.1 ft) to 0.5 m (1.6 ft) below the bottom of the tire chip layer. The control section is located between Wells 1 and 2, so the depth to the groundwater table is about 1.0 m (3.3 ft). Figure 5 shows that the frost penetrated at least 0.4 m (1.3 ft) below the groundwater table. It appears that the location of the groundwater table did not have a significant effect on the depth of frost penetration.

Frost Heave

Frost heave was measured for the winters of 1992 to 1993 and 1993 to 1994. For the 1992 to 1993 season, frost heave was computed as the increase in the elevation of the road surface between December 14, 1992, and February 24, 1993. For the 1993 to 1994 season, computations were made between December 6, 1993, and March 9, 1994. The results are summarized in Figure 8. The sections correspond to those shown in Figure 1. Interpretation of Figure 8 is complicated by the many factors that affect frost heave, including depth to groundwater table, depth to bedrock, and soil type. However, the low heave of the northwest existing road section was probably due to a greater depth to the groundwater table and shallow depth to bedrock. Heave of the northwest transition and the control section was higher because of greater depths to bedrock and shallower depths to the groundwater table. The highest heave was recorded in the southeast transition and southeast existing road section because the groundwater table is very shallow, probably in the range of 1 to 2 m (3 to 6 ft) below the road surface.

The heave was lower in the tire chip sections. This was because of a reduction in the depth of frost penetration by the tire chips. Additionally, the existing road surface was excavated to a level between 152 mm (6 in.) and 475 mm (18 in.) and replaced with tire chips and clean granular fill, as summarized in Table 1. In

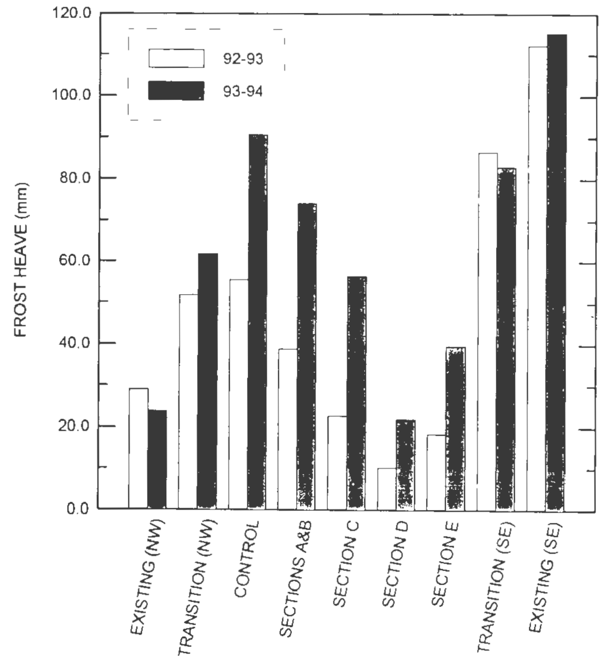


FIGURE 8 Frost heave.

Sections D and E, the heave was reduced by a factor of between 2 and 5 compared with the control section.

Conclusion

A full-scale field trial using tire chips as an insulating layer beneath a gravel-surfaced road showed that tire chips effectively limit the depth of frost penetration. On the basis of tests conducted during the first two winters in service, a 152-mm (6 in.) thick tire chip layer overlain by 305 mm (12 in.) of gravel reduced the depth of frost penetration by 22 to 28 percent. Likewise, a 305-mm (12-in.) thick tire chip layer overlain by either 457 mm (18 in.) or 610 mm (24 in.) of gravel resulted in a 15 to 17 percent reduction in the 1992 to 1993 season and a 33 to 37 percent reduction in the 1993 to 1994 season. In the section with a 305-mm (12-in.) thick tire chip layer covered by 457 mm (18 in.) of gravel, the frost penetrated a maximum of 208 mm (8 in.) into the underlying subgrade soil; in the section with 305 mm (12 in.) of tire chips covered by 610 mm (24 in.) of gravel, the maximum penetration into the underlying subgrade soil was 76 mm (3 in.) Furthermore, the two sections with 305 mm (12 in.) of tire chips experienced a heave of between 10 mm (0.4 in.) and 40 mm (1.6 in.), whereas a nearby control section heaved 55 mm (2.3 in.) to 91 mm (3.6 in.). Clearly, the tire chips significantly reduced the depth of frost penetration and the amount of frost heave.

ACKNOWLEDGMENTS

The authors give their sincere thanks to Nancy Churchill, former Town Manager, and Dick LaChance, Road Commissioner for the town of Richmond, Maine, for volunteering a road in their town for use with the field trial and for obtaining a grant from the state of Maine to pay for its construction. They also thank Jimmy Hayes and Jerry Poole of Pine State Recycling in Nobleboro, Maine, for donating the tire chips used in this project.

REFERENCES

1. Beaton, C. F., and G. F. Hewit. *Physical Property Data for the Design Engineer*. Hemisphere Publishing Corporation, New York, 1989.
2. Jumikis, A. R. *Thermal Geotechnics*. Rutgers University Press, New Brunswick, N.Y., 1977.
3. Humphrey, D. N., T. C. Sandford, M. M. Cribbs, H. Gharegrat, and W. P. Manion. *Tire Chips as Lightweight Backfill for Retaining Walls—Phase I*. Department of Civil Engineering, University of Maine, Orono, 1992.
4. Humphrey, D. N., and T. C. Sandford. Tire Chips as Lightweight Subgrade Fill and Retaining Wall Backfill. In. *Proc., Symposium on Recovery and Effective Reuse of Discarded Materials and By-Products for Construction of Highway Facilities*, FHWA, U.S. Department of Transportation, 1993.
5. *Markets for Scrap Tires*. Report EPA/530-SW-90-047B. Environmental Protection Agency, 1991.
6. Eaton, R. A., and J. Ashcraft. Heavy Weight Deflectometer Results from an Unsurfaced Road Containing Rubber Layers. In *Proc., Fourth International Conference on the Bearing Capacity of Roads and Airfields*, University of Minnesota, Minneapolis, 1994, Vol. 2, pp. 1263–1272.
7. Edil, T. B., and J. P. Bosscher. *Development of Engineering Criteria for Shredded Waste Tires in Highway Applications*. Wisconsin Department of Transportation, Madison, 1992.
8. *Environmental Study of the Use of Shredded Waste Tires for Roadway Subgrade Support*. Groundwater and Solid Waste Division, Minnesota Pollution Control Agency, St. Paul, 1990.
9. Zelibor, J. L. Leachate From Scrap Tires: RMA TCLP Report. In *Proc., Scrap Tire Management Council 1991 Educational Seminar*, Scrap Tire Management Council, Washington, D.C., 1991, pp. 382–391.
10. Humphrey, D. N., and R. A. Eaton. Tire Chips as Insulation Beneath Gravel Surfaced Roads. In *Proc., Second International Symposium on Frost in Geotechnical Engineering*, A. A. Balkema, Rotterdam, 1993, pp. 137–149.
11. Humphrey, D. N., and R. A. Eaton. Tire Chips as Subgrade Insulation—Field Trial. In *Proc., Symposium on Recovery and Effective Reuse of Discarded Materials and By-Products for Construction of Highway Facilities*, FHWA, U.S. Department of Transportation, 1993.
12. Humphrey, D. N. *Specifications—Test Section, Tire Chip Fill as Insulation, Richmond, Maine*. Department of Civil Engineering, University of Maine, Orono, 1992.
13. Bigelow, N., Jr. *Freezing Index Maps of Maine*. Technical Paper 69-5R. Materials and Research Division, Maine State Highway Commission, Augusta, 1969.
14. Gilman, G. D. *The Freezing Index in New England*. Special Report 63. U.S. Army Engineer Cold Regions Research and Engineering Laboratory, Hanover, N.H., 1964.
15. Berg, R., and T. Johnson. *Revised Procedure for Pavement Design Under Seasonal Frost Conditions*. Special Report 83-27. U.S. Army Engineer Cold Regions Research and Engineering Laboratory, Hanover, N.H., 1983.

DESIGN GUIDELINES

Geometric Design of Low-Cost Roads in Developing Countries

Mike McDonald, *University of Southampton, United Kingdom*
Richard Robinson, *University of Birmingham, United Kingdom*

The geometric design process involves selecting the alignment and cross section of a road to meet users' needs. The background and approach to developing a guide for the geometric design of roads in developing countries are described. The approach recognizes that road and driving conditions in developing countries often differ from those in the industrialized countries in terms of (*a*) the traffic mix between commercial vehicles and private cars, as well as between motorized and nonmotorized vehicles; (*b*) the rate and nature of road accidents; and (*c*) the level of economic development and its implications for roads. An iterative approach to design is proposed including the following steps: (*a*) identifying traffic flow, terrain type and road function, leading to a choice of "design class"; (*b*) selecting trial alignments; (*c*) identifying alignment elements that are of a lower geometric standard than that of the chosen design class; and (*d*) estimating approach speeds for the above elements. If they are acceptably consistent, the design goes forward to economic evaluation; if not, the alignment may be amended or the standards relaxed with appropriate measures for safety. The thrust of the approach has been to develop a design methodology that emphasizes the economic aspects of geometric design. The standards recommended tend to be lower than many of those in common use. Recommended standards include maximum carriageway widths of 6.5 m with shoulders for use by nonmotorized traffic, roads with 5-m carriageways to carry up to 400 vehicles per day where 1.0-m shoulders may be used for passing, roads of 2.5 to 3.0 m for very low flows of traffic, with room for passing, and horizontal radii as low as 15 m. The guide itself was published by the Transport Research Laboratory as Overseas Road Note 6.

The geometric design process involves selecting the alignment and cross section of a road to efficiently meet the needs of the users. Predetermined design standards are often used. These standards are intended to satisfy two interrelated objectives:

1. To provide acceptable levels of safety and comfort for drivers, and
2. To provide efficient and economic design.

Standards in Developing Countries

Historically, geometric design standards used in developing countries have been based on those in industrialized countries. A study carried out by the then Transport and Road Research Laboratory (TRRL) (*1*) considered the application of the American (*2*), Australian (*3*), and British (*4*) standards to developing countries. The study concluded that roads and driving conditions in developing countries were sufficiently different from those in industrialized countries to merit further consideration. In particular, differences were identified in the following significant areas:

- The traffic mix between commercial vehicles and private cars as well as often between motorized and nonmotorized vehicles,
- The rate and nature of road accidents, and
- The level of economic development and the implications of this development on the function of roads being provided.

As a result of this study, the U.K. Overseas Development Administration (ODA) commissioned Roughton and Partners to carry out further research and studies. The goal was to produce guidance on geometric design specifically for low-volume roads in developing countries. The results of the studies were published by Boyce et al. (*5*), and the resulting design guide was issued as TRRL Overseas Road Note 6 (*6*). This paper is based on the results of that work.

The costs of road construction can be substantial. In developing countries, it is particularly important that economic solutions be found. However, in many developed countries, design standards have been set on the basis of the need to ensure safety, which has often led to high-cost designs, even though the precise relationship between the high standards and accidents has not been established. The approach adopted for ODA has concentrated on the development of a design process that emphasizes the economic aspects of geometric design.

Approach to Design

General

Drivers are usually provided with safety and comfort by development of a consistent alignment so that they do not face an unexpected change. This alignment includes adequate sight distances for the prevailing speeds and road surfaces, road space for vehicle maneuvers, and clear signing and road marking. A road on which a driver can see ahead a sufficient distance to stop safely is likely safe for other road users. Segregation of road users with different characteristics and objectives is also beneficial to safety.

Efficient design requires that the costs of construction match the level of expected benefit. As construction costs increase when a road alignment is made straighter and wider with reduced gradients, the additional benefits must also increase. Economic benefits can be expected in reduced vehicle operating costs, savings in travel time, and reductions in accidents. The latter two benefits assume that values of travel time and accidents can be determined and that the effects of variations in geometric design on accidents can be predicted.

A key issue in the application of design standards is the interaction between such standards and the characteristics of driver behavior, particularly speed.

Design Standards

Design standards can provide an essential base for decision making if they are applied with appropriate understanding of economics and flexibility. The essence of the process is to develop roads that meet a functional objective closely related to the level of traffic. The standard will be related to traffic volume and characteristics, terrain, and the function of the road. Potential hazards must be identified at an early stage and treated in the geometric design process.

The recommended road standards shown in Tables 1 and 2 are linked by design speed, which varies with terrain and design class and level of flow. A mountainous terrain with a low level of traffic would have a lower design speed. However, design speeds arbitrarily linked to function are not the basis for design decisions.

Standards for alignment use the minimum values normally allowed. However, in many situations, terrain and other circumstances are such that minimum values need never be applied and link speeds are substantially above design speed values. The most economic designs often do not involve the use of minimum standards. Levels of traffic may be such that the benefits gained from wider, straighter, shorter roads may offset the necessary extra construction costs. To ensure safe operation, the final alignment must incorporate additional procedures into the design process. These procedures are discussed in the following section.

Design Process

An outline of the design process, which is intended to result in sound economic design, is shown in Figure 1. The design involves the following steps:

1. Traffic flow, terrain type, and road function are defined, and a design class is chosen;
2. Trial alignments are selected (a road consists of discrete geometric elements, contiguous groups of which are combined to form sections; design is undertaken over sections with minimum lengths of about 1 km;
3. Elements of lower geometric standard are identified and compared with the standards of the design class chosen; and
4. Estimates of approach speeds are made for the geometric design elements identified above; if they are consistent, the design goes forward to economic evaluation; if not, the road alignment may be amended or

TABLE 1 Road Standards

ROAD FUNCTION	DESIGN CLASS	TRAFFIC FLOW* (ADT)	SURFACE TYPE	WIDTH (m)		MAXIMUM GRADIENT (%)	TERRAIN/DESIGN SPEED (km/h)		
				CARRIAGE-WAY	SHOULDER		MOUNTAINOUS	ROLLING	LEVEL
Arterial	A	5,000-15,000	Paved	6.5	2.5	8	85	100	120
	B	1,000-5,000	Paved	6.5	1.0	8	70	85	100
	C	400-1,000	Paved	5.5	1.0	10	60	70	85
Collector	D	100-400	Paved/ Unpaved	5.0	1.0+	10	50	60	70
Access	E	20-100	Paved/ Unpaved	3.0	1.5+	15	40	50	60
	F	< 20	Paved/ Unpaved	2.5/3.0	Passing Places	15/20	N/A	N/A	N/A

* The two way traffic flow is recommended to be not more than one Design Class step in excess of first year ADT.

\+ For unpaved roads where the carriageway is gravelled, the shoulders would not normally be gravelled: however, for Design Class D roads, consideration should be given to gravelling the shoulders if shoulder damage occurs.

TABLE 2 Speed-Related Design Parameters

DESIGN SPEED (km/h)	STOPPING SIGHT DISTANCE (m)	MINIMUM CURVATURE VALUES					MINIMUM SAFE OVERTAKING SIGHT DISTANCE (m)*
		HORIZONTAL (m)		VERTICAL CURVES (m)			
		PAVED (10% SUPERELEVATION)	UNPAVED (ZERO SUPERELEVATION)	CREST K TO OBJECT ON ROAD	CREST K TO ROAD SURFACE	SAG K FOR COMFORT	
Two Lane							
120	230	450	-	120	250	22.6	590
100	160	320	-	60	125	13.1	430
85	120	210	-	30	70	8.1	320
70	85	130	190	16	35	4.8	240
60	65	85	125	10	20	3.5	180
50	50	60	80	5	11	2.2	140
40	35	30	40	3	6	1.3	Not applicable
30	25	15	20	1.5	3	0.7	Not applicable
Single Lane							
60	130	85	125	25	20	3.5	
50	100	60	80	15	11	2.2	
40	70	30	40	7	6	1.3	
30	50	15	20	4	3	0.7	

* These values are the normal minimum assuming that an overtaking vehicle may safely abandon the manoeuvre if an opposing vehicle comes into view. The values should be available continuously in all places where overtaking is permitted.

Note : The following assumptions have been made in calculating the above :

. Reaction time of 2 sec.
. Eye height of 1.05m. Object height of 0.2m for stationary object on the road and 1.05m for approaching vehicle. (Zero object height values have been included for use where it is necessary to see the road surface, eg. approaching a ford or drift.) The values for single lane roads have been based on the assumption that approaching vehicles should be able to stop safely before colliding.
. The following values of side and longitudinal friction factor were taken to estimate acceptable values of horizontal curvature for both paved and unpaved conditions.

Design speed (km/h)	120	100	85	70	60	50	40	30
Side friction factor	0.15	0.15	0.18	0.20	0.23	0.25	0.30	0.33
Longitudinal friction factor	0.33	0.37	0.40	0.43	0.47	0.50	0.55	0.60

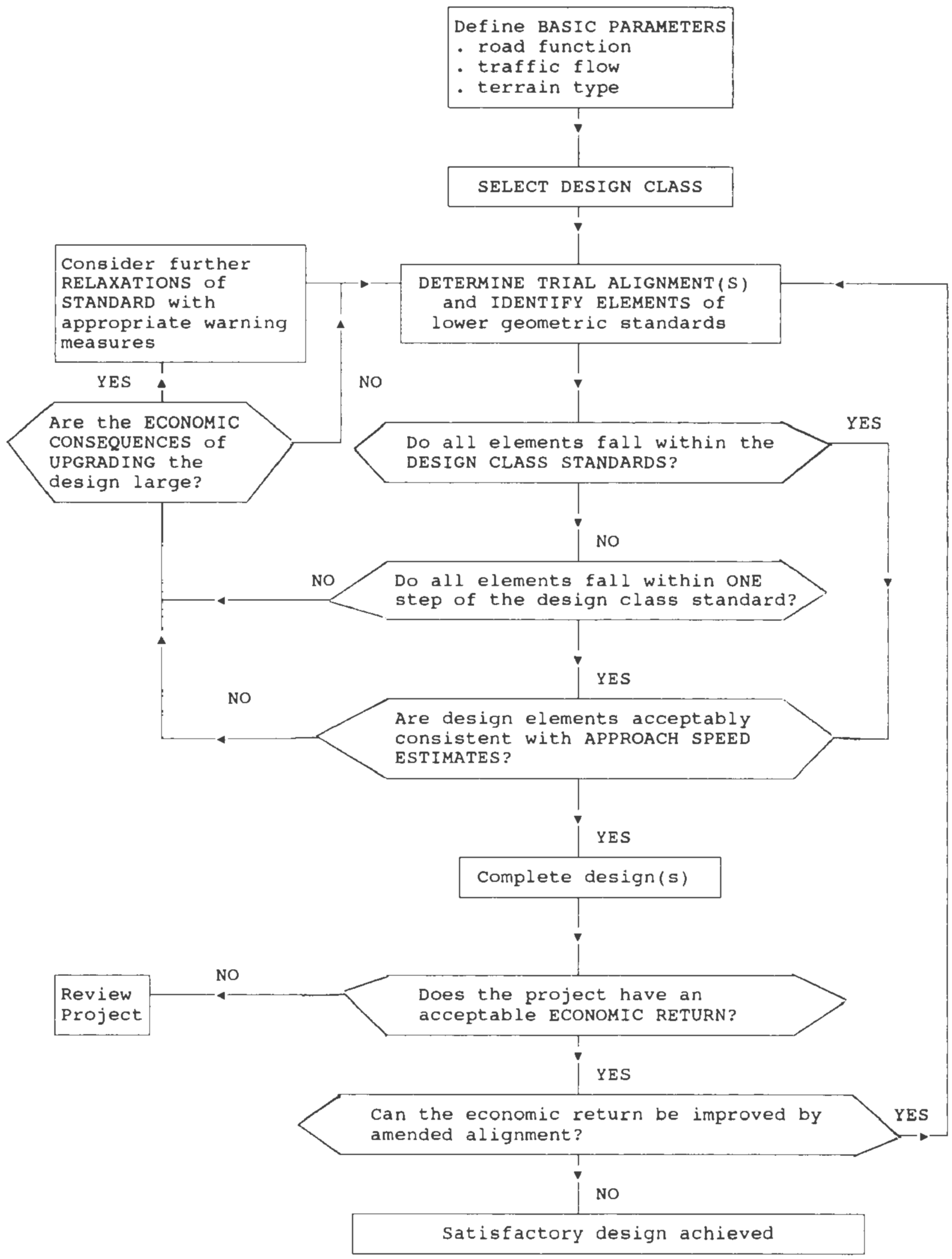


FIGURE 1 Design process.

the standards relaxed with the appropriate measures taken for safety.

Road Function

Each interurban road may be classified as arterial, collector, or access. Figure 2 and Table 1 demonstrate this classification.

Arterial roads are the main routes connecting national and international centers. Traffic on these roads is generated at the urban centers and from interurban areas through the collector and access road systems. Trip lengths are usually relatively long and levels of traffic and speed relatively high. Geometric standards must enable efficient traffic operation under these conditions because vehicle-to-vehicle interactions may be high.

Collector roads link traffic to and from rural areas, to adjacent urban centers, or to the arterial road network. These roads have intermediate traffic flows and trip lengths; the need for high geometric standards is therefore less important.

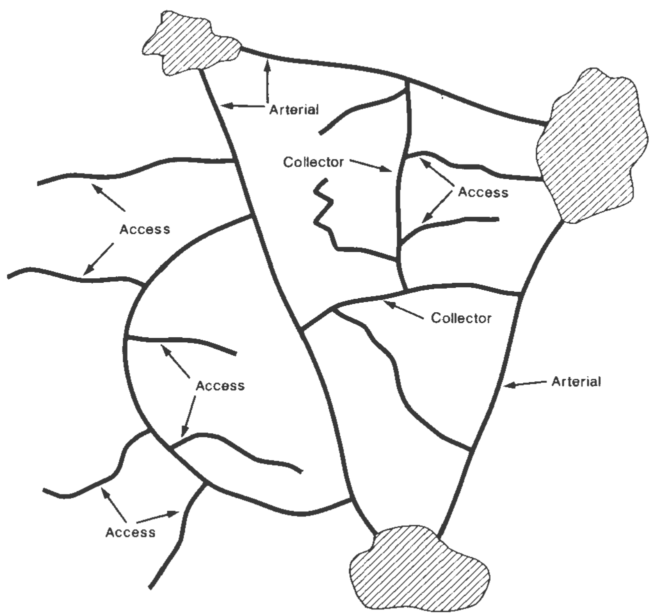


FIGURE 2 Road hierarchy and function.

Access roads are the lowest level in the network hierarchy. Traffic is light and aggregated in the collector road network. Geometric standards may be low and need only provide appropriate access to the rural agricultural, commercial, and population centers served. Most of the total movements will be nonmotorized traffic.

The hierarchy shown in Figure 2 will have many overlaps of function, and clear distinctions will not always be apparent on functional terms. This hierarchy should not be confused with the division of administrative responsibilities, which may be based on historic conditions.

It is inappropriate to design the lowest design class of road on the basis of geometric standards. The sole criterion of acceptability is the achievement of an appropriate level of access. In these situations, design should be based on minimum values of the radius, width, and gradient for the passage of a suitable design vehicle.

Design Flow

Within the functional hierarchy, traffic is aggregated as it moves from access to collector to arterial road, and levels of flow are normally correlated with road type. However, flow levels vary between countries and regions. Designation of a road by functional type should not lead to overdesign for the levels of traffic actually encountered. Designs that are not cost-effective reduce the likelihood that roads will be built, which results in wasting important national resources.

Design Classes A to F have associated bands of traffic flow, as shown in Table 1. The range of flows extends from fewer than 20 to more than 15,000 motorized vehicles per day, excluding motorcycles, and covers the design conditions for all single-carriageway roads.

The levels of flow at which design standards change are based on the best evidence available. However, the somewhat subjective boundaries should be treated as approximate to account for the uncertainties inherent in traffic estimation and economic variability. Therefore, design flows should normally be constrained to no more than one design class step higher than the annual average daily traffic (ADT) in the first year. A road with a first-year traffic flow of 390 vehicles per day rising to 1,100 vehicles per day should be constructed to Design Class C rather than Design Class B geometry (see Table 1). The design flow band in this case is 400 to 1,000 vehicles per day. Design to the higher Design Class would result in an overdesigned facility during most of the life of the road and could provide a solution that was not cost-effective. If the initial flow were 410 vehicles per day, design would still be to Design Class C. It is particularly important that roads not be overdesigned on the basis of high traffic growth rates, which are normally uncertain in developing countries.

Composition

In some situations, heavy vehicles have a greater effect on congestion than light vehicles. However, no attempt has been made to use passenger car unit (pcu) equivalent values, as these can vary substantially with composition and conditions. The relative effects of heavier vehicles vary with level of flow, geometry, and vehicle performance, and well-researched, consistent values are not available for the range of flows covered in this design guide. All flows are presented as ADT values. However, high percentages of heavy vehicles in a traffic stream may require consideration of enhanced standards, particularly carriageway width standards.

Capacity

Congestion increases with increased traffic flow when there is no safe passing opportunity. The result is long journey times, increased vehicle operating costs, and sometimes more accidents as frustrated drivers take risks.

Practical capacity is usually estimated to have been reached when the level of congestion becomes "unacceptable." Capacity is affected by increased proportions of heavy vehicles, greater unevenness in directional flows, reduced passing opportunities, animal-drawn vehicles, and pedestrian activity. Normally acceptable practical capacity is about 1,500 to 2,000 vehicles per hour. This may be increased substantially by the provision of short sections of climbing and passing lanes.

Terrain

A simple classification of "level," rolling," or "mountainous" has been adopted and is defined by subjective description and the average ground slope. The average ground slope is measured as the number of 5-m contour lines crossed per kilometer on a straight line that links the two ends of the road section. However, where the corridor for the road is already known, counting contours within the corridor could lead to a more appropriate terrain classification. Definitions of the classification terms follow.

- **Level** (0 to 10 five-meter ground contours per kilometer): Level or gently rolling terrain with largely unrestricted horizontal and vertical alignment. Minimum values of alignment are seldom necessary. Roads follow the ground contours for the most part; amounts of cut and fill are very small.
- **Rolling** (11 to 25 five-meter ground contours per kilometer): Rolling terrain with low hills introducing moderate levels of rise and fall with some restrictions on vertical alignment. While low-standard roads can follow the ground contours with small amounts of cut and fill, the higher standards require more substantial amounts of cut and fill.
- **Mountainous** (more than 25 five-meter ground contours per kilometer): Rugged, hilly, and mountainous with substantial restrictions in both horizontal and vertical alignment. Higher-standard roads generally require large amounts of cut and fill.

In general, construction costs are greater as the terrain becomes more difficult. Higher standards become less justifiable or achievable in these situations than for roads in flat or rolling terrain. Drivers should also expect lower standards in such conditions and adjust their driving accordingly to minimize the risk of accident. Design speed therefore varies with terrain.

Cross Section

Roads should be wide enough to safely and efficiently carry traffic but no wider than necessary to minimize cost of construction and maintenance. Recommended values are given in Table 1, and typical cross sections are shown in Figure 3.

For access roads with volumes of traffic lower than 100 ADT, single-lane operation is adequate since there is small probability that vehicles will meet. Passing can be achieved at reduced speeds in designated passing zones or on shoulders. If sight distances are adequate for safe stopping, cars can pass without hazard, and the overall loss in efficiency brought about by the reduced speeds will be small. It is not cost-effective to widen the running surface in such circumstances—a basic width of 3.0 m is normally sufficient. In some situations, even 2.5 m is adequate.

On roads with traffic volumes of 100 to 1,000 ADT, the amount of passing increases and pavement widening becomes worthwhile operationally and economically. However, with the generally high cost of capital for construction in developing countries and the relatively low cost of travel time, reductions in speed when approaching vehicles pass remains acceptable for such traffic levels. Running surface widths of 5.0 and 5.5 m are recommended. For arterial roads with traffic volumes of more than 1,000 ADT, a running surface 6.5 m wide will allow vehicles in the opposite direction to pass safely without needing to move laterally in their lanes or slow down.

Shoulders are recommended for all but the lowest design class, and these should normally be paved when the carriageway is paved. Shoulders are intended to perform three main traffic functions:

- To provide additional maneuvering space on roads of low functional classification and traffic flow,

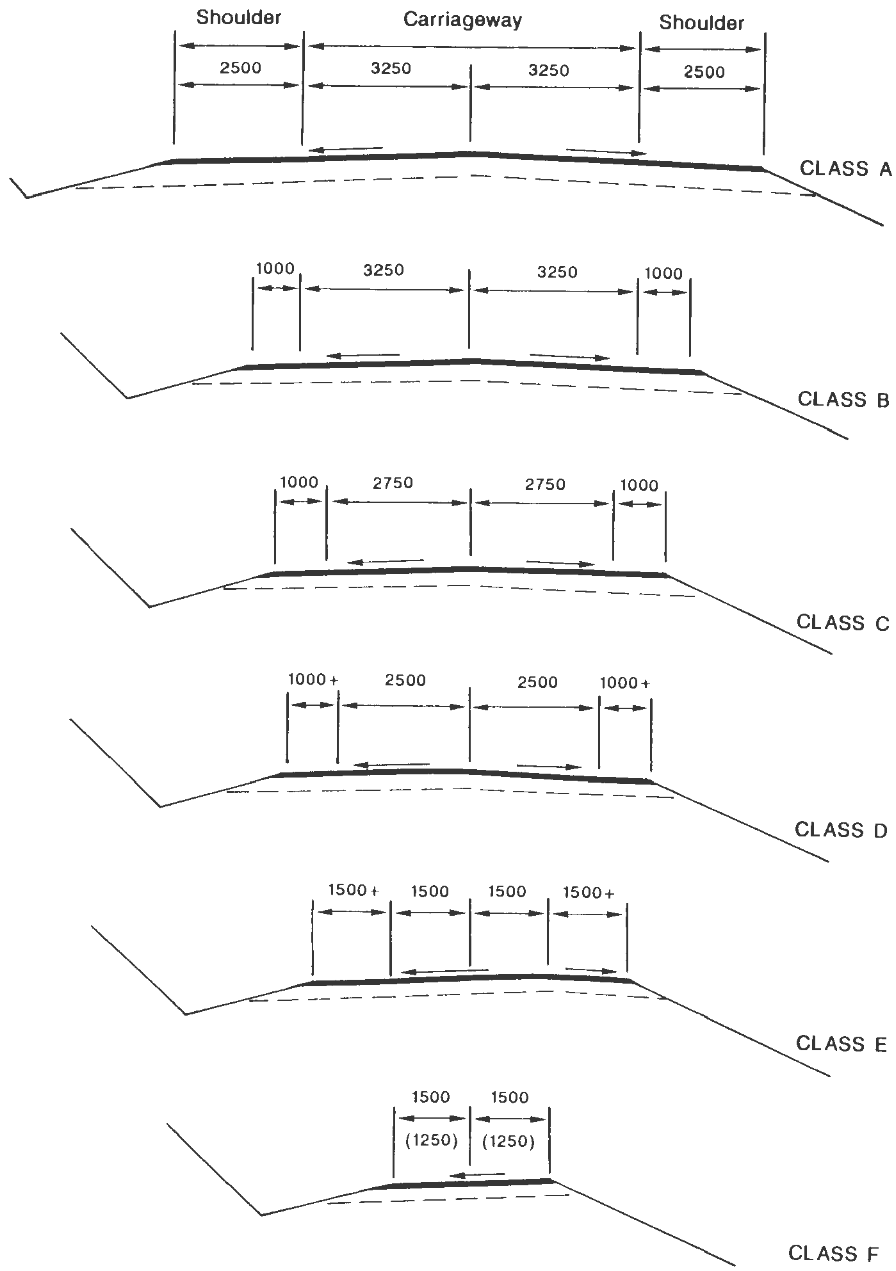


FIGURE 3 Typical cross section.

- To provide parking space at least partly off the carriageway for vehicles that have broken down, and
- To enable nonmotorized traffic to travel with minimum encroachment on the carriageway.

Additionally, it may be desirable to provide sufficient width for two-way movement during road work.

The lowest design class with a width of 3.0 (2.5) m is not adequate for passing and overtaking. Passing zones must be provided. The increased width in such zones should be enough to allow two trucks to pass (i.e., a minimum of 5.0 m total width). Vehicles would be expected to stop or slow to a very low speed.

Normally, passing zones should be located every 300 to 500 m, depending on the terrain and geometric conditions. Sight distances, the likelihood of vehicles meeting between passing zones, and the potential difficulty of backing up should be considered. In general, passing zones should be constructed at the most economic locations as determined by terrain and ground condition—such as transitions from cut to fill—rather than at precise distance intervals.

The length of individual passing zones will vary with local conditions and the sizes of vehicles in common use. Generally, a length of 20 m, including tapers, is sufficient for most commercial vehicles on these roads.

A clear distinction should be drawn between passing zones and lay-bys. Lay-bys may be provided for specific purposes, such as parking or bus stops, to allow vehicles to stop safely without impeding through traffic.

Safety

The operating conditions on roads in developing countries are normally very different from those in developed countries. Principal areas of difference are the substantial variations in vehicle performance and condition, the often large amounts of nonmotorized traffic, and the low levels of training and control of road users.

Road accident rates in developing countries are high and result in substantial economic loss as well as pain, grief, and suffering. However, in view of the uncertainties of accident prediction, it has not been possible to evaluate the specific effects of the geometric design parameters recommended in this guide on accident rates. Therefore, accident rates must be monitored carefully to identify the need for specific remedial treatment and to form a basis for future local amendments to the design procedure.

In general, designers should be aware of the need to consider safety. Designers should take advantage of opportunities during design or construction to provide substantial benefits at little additional cost. The following factors should be considered when designing for safety:

1. Nonmotorized traffic should be segregated by physical barriers such as raised curbs as much as possible. Designs should include features to reduce speeds in areas of significant pedestrian activity, particularly at crossings.
2. To minimize the effect of a driver who has lost control and left the road, the following steps should be taken:
    a. Steep open side drains should be avoided since these increase the likelihood that vehicles will overturn; trees should not be planted immediately adjacent to the road.
    b. Because of their high costs of installation and maintenance, guardrails should only be introduced at sites of known accident risk.
    c. Junctions and accesses should be located where full safe stopping sight distances are available.

A checklist of engineering design features that affect road safety is given in Figure 4.

Conclusion

The objective of this study was to review existing design standards and methods and make recommendations for designs in developing countries. Many aspects of design standards are based on good practice, and there is little hard evidence to link particular features such as width of cross section to safety. The application of standards over long periods has meant that little evidence is available to compare alternatives. For example, the standard 7.3-m single-carriageway road width in the United Kingdom is a direct metrication of the 24-ft standard, which has been used for more than 50 years. In all probability, a 6.5-m carriageway would perform as safely and save considerable costs. The additional length of new construction that could be incorporated for the same total funds could result in an overall accident saving. For developing countries, more results are becoming available that indicate the boundaries at which increased risk becomes significant. These results formed the basis for the recommended standards (*5*). Key design features for optimum economic return include the following:

1. Maximum carriageway widths of 6.5 m with shoulders designed to reflect use by nonmotorized traffic,
2. Roads with 5-m carriageways for flows of up to 400 vehicles per day where the 1.0-m shoulders may be used for passing,
3. Roads of 2.5 to 3.0 m width for very low flows with passing zones, and
4. Horizontal radii as low as 15 m.

However, side friction factor varies with speed. The highest speed is lower than it is in some developed country standards, which better reflects recent research results.

Design guidance often results in designs that are of a lower geometric standard than those previously used. However, the economic return will be greater, and there is no evidence that higher standards are significantly safer.

A major issue in the selection of a design standard is the design flow; in developing countries, traffic growth can fluctuate substantially. These recommendations tend toward lower standards where future flows are uncertain. In developing countries, excess expenditure on overly ambitious geometric standards can result in the removal of key resources from other sectors of the community. Also, the history of maintenance indicates that too much expense on new construction has limited the budget available for maintenance. The often resulting reductions in surface roughness are usually much more significant economically than a shorter, straighter alignment is.

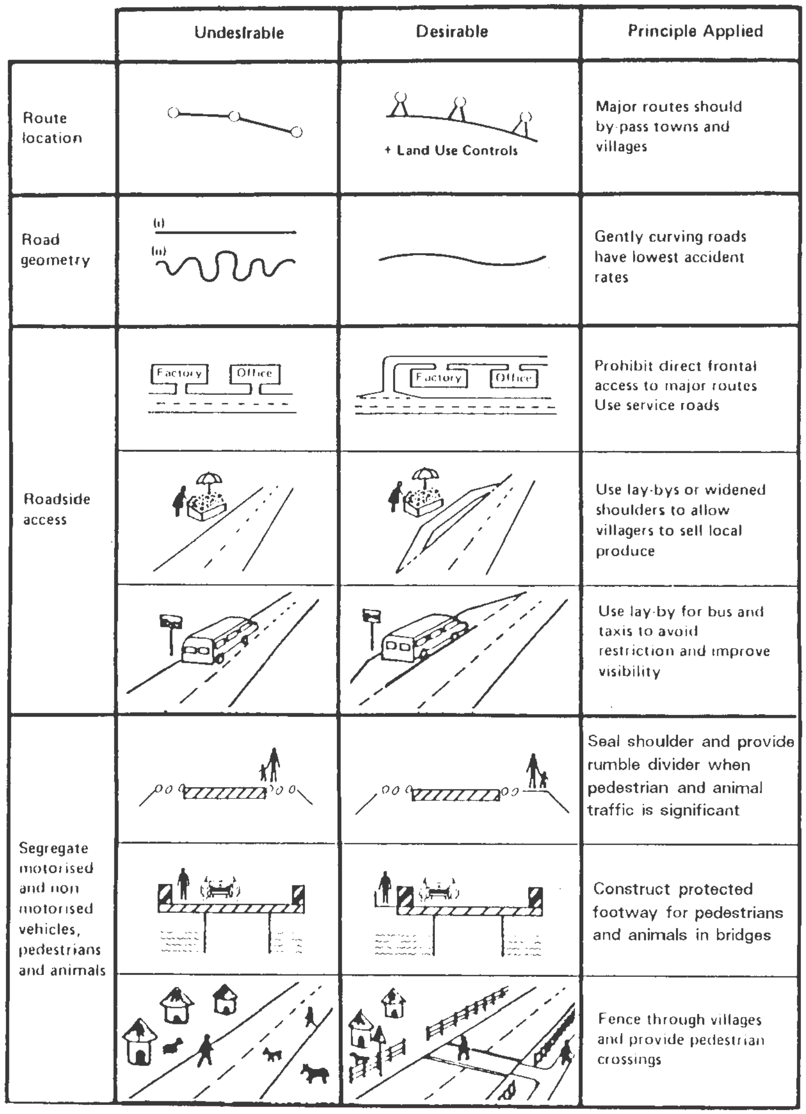


FIGURE 4 Checklist of engineering design features affecting road safety.

ACKNOWLEDGMENTS

The research described was undertaken by ROUGHTONS, Consulting Engineers, under contract to the Overseas Unit of TRL.

REFERENCES

1. Kosasih, D., R. Robinson, and J. Snell. *A Review of Some Recent Geometric Road Standards and Their Application to Developing Countries.* TRRL Research Report 114. Transport and Road Research Laboratory, Crowthorne, England, 1987.
2. *A Policy on Geometric Design of Highways and Streets.* AASHTO, Washington, D.C., 1984.
3. *Interim Guide to the Geometric Design of Rural Roads.* National Association of Australian State Road Authorities, Sydney, 1980.
4. *Highway Link Design.* Departmental Standard 9/81. U.K. Department of Transport, London, 1981.
5. Boyce, A. M., M. McDonald, M. J. Pearce, and R. Robinson. *A Review of Geometric Design and Standards for Rural Roads in Developing Countries.* Contractor Report 94. Transport and Road Research Laboratory, Crowthorne, England, 1988.
6. TRRL Overseas Unit. *A Guide to Geometric Design.* Overseas Road Note 6. Transport and Road Research Laboratory, Crowthorne, England, 1988.

Lowell Test Road: Helping Improve Road Surfacing Design

Mark A. Truebe and Gary L. Evans, *USDA Forest Service, Eugene, Oregon*
Pete Bolander, *USDA Forest Service, Portland, Oregon*

The primary objective of this study was to determine the effects of tire pressure, aggregate thickness, and aggregate quality on surfacing performance in the context of verifying the Surfacing Thickness Program (STP). STP is a new aggregate thickness design model that has been adapted from previous U.S. Army Corps of Engineers research. Secondary objectives were to determine the relationship between surface and subgrade rutting and the implications for road maintenance, as well as to evaluate the use of the dynamic cone penetrometer (DCP) as a tool for evaluating material strengths. The study was accomplished by constructing, testing, and monitoring 18 test sections in western Oregon in 1992 and 1993. Tests were conducted from January through April to obtain the effect on log haul during normal wet weather periods. Surface and subgrade rutting was measured using a pressure transducer, which gave digital elevation results. The test sites were subjected to unloaded and loaded log trucks that were varied as to high and low tire pressures. The major conclusion of the study is that the STP accurately predicts aggregate surfacing rutting. Additional findings include the following: (*a*) surface rutting is primarily due to densification and aggregate shear, and only a small portion of the rut is observed in the subgrade; (*b*) the DCP is a useful tool for rapidly evaluating material strength properties; and (*c*) central tire inflation (CTI) showed less rutting than highway tire pressure.

The USDA Forest Service road network consists of 594 000 km (369,000 mi), approximately 65 percent of which is unsurfaced, 30 percent is aggregate surfaced, and 5 percent is paved (D. Badger, unpublished data). Annual road construction consists of 1930 km (1,200 mi) per year.

Road use, especially during periods of wet weather, causes accelerated surface deterioration (rutting) and may result in surfacing erosion and sedimentation. The Forest Service has attempted to reduce road maintenance costs and surfacing sedimentation (*1–3*; Copstead, unpublished data; Ashmore, unpublished data; Foltz unpublished data). The Lowell Road Test is a part of this effort to evaluate design, operational, and maintenance effects on surfacing requirements and sedimentation.

Objectives

The objectives of this study include the following:

1. Determining the effects of tire pressure, aggregate thickness, and aggregate quality on surfacing performance in the context of verifying the Surfacing Thickness Program (STP), a new aggregate thickness design model (*1*);

2. Determining the relationship between surface and subgrade rutting and its implications for road maintenance; and
3. Evaluating the use of the dynamic cone penetrometer (DCP) as a tool for evaluating material strengths.

Scope

This study was accomplished by constructing and monitoring the performance of 18 test sections on Road Number 1821190 in the Lowell Ranger District of the Willamette National Forest in Oregon. A plan view of the test section layout is shown in Figure 1.

The test sites were subjected to unloaded and loaded log trucks traveling uphill and downhill, respectively. Specific sites were subjected to either high or low tire pressures with specially equipped central tire inflation (CTI) trucks.

Field testing consisted of in situ California bearing ratio (CBR), DCP, and moisture and density tests. Site monitoring consisted of periodically measuring surface and subgrade rutting, records of traffic, and maintenance records.

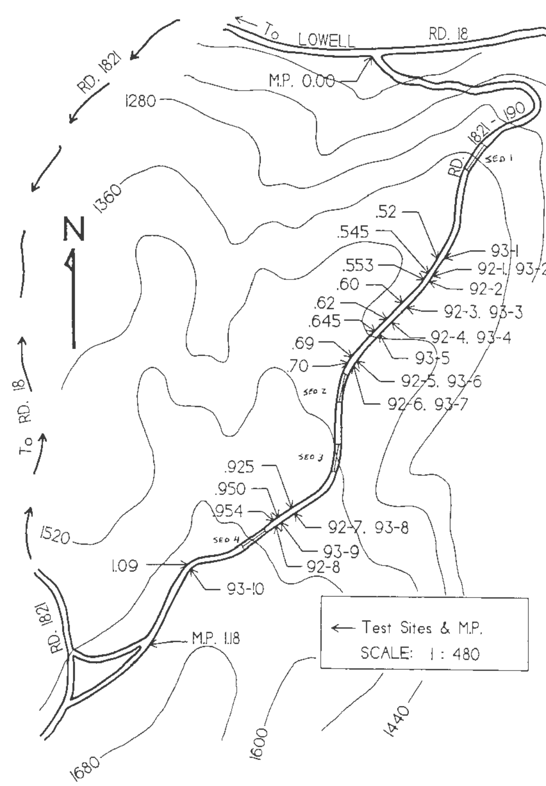


FIGURE 1 Site map.

Surfacing Thickness Program

STP is the Forest Service surfacing thickness design computer program for native- and aggregate-surfaced roads; it was first released in 1989. Whitcomb et al. (*1*) provided details on the selected model and incorporated a user's guide for STP. Yapp et al. (*4*) discuss the selection of the surfacing thickness design model for the computer from a variety of models used by the Forest Service before 1989.

Model

The Forest Service surfacing thickness design model is based on the U.S. Army Corps of Engineers 1978 Barber (*5*) equation. The model, shown below, assumes that the top layer has a higher strength than the bottom layer ($C_1 > C_2$):

$$RD = \frac{0.1741 P_k^{0.4704} t_p^{0.5695} R^{0.2476}}{(\log t)^{2.002} C_1^{0.9335} C_2^{0.2848}}$$

where

RD = rut depth (in.)
P_k = equivalent single-wheel load (ESWL) (kips),
t_p = tire pressure (psi),
t = thickness of top layer (in.),
R = repetitions of load or passes,
C_1 = in situ CBR of top layer, and
C_2 = in situ CBR of bottom layer.

This equation indicates the thickness necessary to limit the rut depth for known traffic conditions and layer strengths. This thickness applies to aggregate-surfaced roads designed by the Forest Service, on which the average rut depth is limited to 50 mm (2 in.). For native-(earth) surfaced roads, the model is used to determine the expected rut depth for known traffic conditions and layer strengths with an assumed top layer thickness of 152 mm (6 in.).

Initial STP

As developed in 1989, STP performs these calculations using interactive computer screens on an IBM-compatible personal computer. STP allows for the following:

1. Up to two periods (seasons) of layer strengths for each native-surfaced road and up to four seasons for each aggregate-surfaced road,
2. User-determined reliability based on work by Barber et al. (*5*), and
3. Economic analysis modules to determine the life-cycle cost of the proposed facility.

Current STP

From 1989 to 1993, STP was modified to include the following:

1. A module to determine the additional thickness for an existing aggregate-surfaced road and
2. A module to calculate the allowable ESWLs based on a known thickness of the top layer and the layer strengths for a given rut depth.

Future STP

Whitcomb et al. (*1*) emphasized the need for field studies to refine the design algorithm, mainly because of its need for field verification. This is one of the primary objectives of this study. At the Waterways Experiment Station (*3*) the U.S. Army Corps of Engineers has performed validation that has resulted in a slight modification to the coefficients in the rut-depth model.

The Forest Service is continuing to evaluate these validation studies and will modify the design model as necessary. The Forest Service is currently preparing a guide that will summarize this information entitled the *Surfacing Design Guide for Low Volume Roads*, which will serve as the latest user's guide for STP and provide comprehensive guidelines on surfacing selection, surfacing design, and use of STP as a maintenance tool for native- and aggregate-surfaced roads.

Test Section Design

Site Characteristics

The test site consisted of a 6.9-km (4.3-mi) loop of an existing Forest Service road with a gate to allow exclusive use for this test. The road grades varied from 7 to 14 percent, and the test sections were approximately 61 m (200 ft) long to allow for site instrumentation and provide a transition zone for different aggregate thicknesses. The test sections were situated on tangents to reduce the likelihood of vehicle braking or accelerating on the test site. Vehicle speeds were less than 48 km (30 mi) per hour.

The particular site variables selected for study included aggregate quality, strength and thickness, subgrade strength, and tire pressure. Table 1 shows how the above variables were included in the design for road tests of 1992 and 1993.

Aggregate Materials

Rut depths were measured on sections surfaced with good-quality and marginal-quality aggregates. Rutting is dependent upon material strength measured by CBR. For an aggregate material, the CBR is dependent on gradation and quality.

The good-quality aggregate met all of the requirements of the Forest Service Standard Specifications. These include sieve requirements for a dense, 25.4-mm (1-in.) minus gradation, Los Angles Abrasion (AASHTO T96) percent wear of 40 maximum, durability index (AASHTO T210) of 35 minimum, and a sand equivalent (AASHTO T176) of 35 minimum. The poor-quality aggregates had sand equivalent and durability (fines) values less than 35, with marginal test results in Oregon air degradation (OSHD TM 208-86).

The specified good-quality aggregate that was placed on Sites 1, 2, 5, and 9 in 1993 had good durability properties but poor gradation characteristics. The aggregate was open-graded, which resulted in an inability to attain the desired compaction. The gradation was improved by blending sand-sized materials with the surfacing rock in the field with limited success, as shown by low CBR values obtained for a good-quality aggregate.

In 1992 the average field densities of the subgrade generally ranged from 95 to 100 percent of AASHTO T-99 density, except for Site 1992-8, where the density was about 85 percent.

Subgrade Testing

The subgrade soils generally had a Uniform Soil Classification System grade of MH or ML. Typical values of in situ dry density and moisture content were 1280 kg/m^3 (80 lb/ft^3) and 35 percent, respectively. Additional subgrade soil values were determined by laboratory CBR, field CBR, and DCP tests. Field testing occurred before any traffic and at later points in the program. The field CBRs were performed in excavations in the wheel tracks between the monitoring lines. The DCP testing was done nearby, in the wheel tracks, without excavating any material.

Construction and Operational Plan

Construction

In 1992 construction of the test sites was accomplished by first blading off the existing aggregate from selected sites on Road 1821190. The desired thickness of either good- or poor-quality aggregate was then placed on the subgrade. Once the aggregate was placed, a 0.6-m (2-ft) wide trench was excavated transverse to the roadway to the top of the subgrade for placement of the subgrade tube. Typical sites are shown in Figures 2 and 3.

TABLE 1 Test Site Characteristics and Results

1992										
Site	1	2	3	4	5	6	7	8		
Aggregate Quality	Good	Good	Good	Good	Poor	Poor	Good	Good		
Ini. Thickness	140	183	234	269	221	211	160	132		
Surfacing CBR	50	50	21.5	37.5	45	32.5	58	23		
Subgrade CBR	17	17	11	19.5	22.5	14	16	6		
Tire Pressure	Std	Std	Std	Std	Std	Std	CTI	CTI		
Surface Rut	120	25	20	10	28	20	46	110		
Subgrade Rut	10	5.1	0.0	0.0	5.1	2.5	2.5	120		
Thick. Change	-4.6	-5.1	-10	-13	-41	-48	-33	-74		
R Squared	.938	.891	.203	.389	.800	.559	.965	.710		
Note: R Squared between actual and predicted rut										
1993										
Site	1	2	3	4	5	6	7	8	9	10
Aggregate Quality	Good	Good	Good	Good	Good	Poor	Poor	Good	Good	Poor
Ini. Thickness	150	142	211	234	221	206	188	147	165	175
Surfacing CBR	23.5	20.5	86	60	28	50	74	61	18	33.5
Subgrade CBR	6.5	8.5	17.7	19.5	8.5	13.5	16	20.5	15	8
Tire Pressure	Std	Std	Std	Std	Std	Std	Std	CTI	CTI	CTI
Surface Rut	81	99	5.1	10	41	12.7	25	23	135	132
Subgrade Rut	7.6	7.6	0.0	2.5	17.8	5.1	7.6	5.1	15.2	5.1
Thick. Change	27.9	-12.7	-10	7.6	-7.6	-7.6	-7.6	-5.1	-51	41

note 1) Surface and Subgrade ruts are maximum obtained and may occur at different traffic levels for different sites.
2) Positive thickness change occurred where additional aggregate had been added. 25.4 mm = 1 inch

In 1993 additional test sites were added in a similar manner with the following exceptions. The subgrade tubes were placed in a 38-mm (1 1/2-in.) deep notch excavated in the subgrade before aggregate placement. This notch was used to ensure that no movement of the tube occurred in the direction parallel to the traffic, as had occurred on some sites in the Weyerhauser test road (Copstead, unpublished data) and the Lowell 1992 test.

Surface and Subgrade Deformations

The deformations of the top of the road surface and subgrade were monitored by means of a specially designed pressure probe, which is shown in Figure 4. The device consists of a 0- to 13.8-kPa (2-psi) pressure transducer sealed in a 13-mm-diameter by 76-mm (1/2-in.-diameter by 3-in.) section of conduit with silicon and epoxy. The conduit has two 6.3-mm (1/4-in.) diameter tubes (one vented to air and the other filled with an ethylene glycol solution), an electrical cable, and a wire line extending from one end. The conduit is fed into a 25.4-mm (1-in.) inside-diameter hydraulic hose that was buried in the top of the subgrade with the ends extending out of the ground into the ditch line. Additional information on this measuring device can be found in other sources (6).

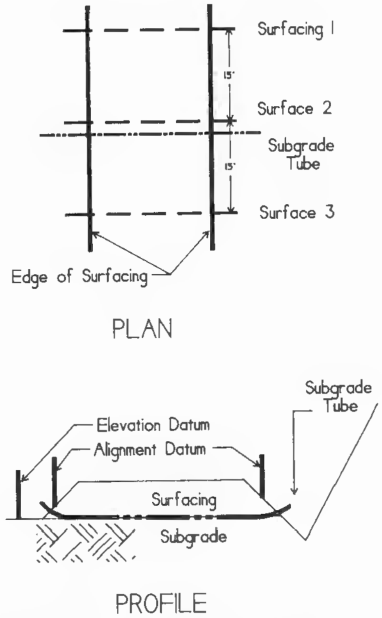


FIGURE 2 Typical site plan.

Elevation profiles of the subgrade were obtained by connecting the liquid-filled tube to an elevation datum, connecting the cable to a specially designed data logger, and pulling the probe through the hydraulic hose. The probe was stopped at 152-mm (6-in.) intervals to obtain elevation measurements. Surface profile data were obtained by simply stopping the probe at 152-mm (6-in.) increments in the line across the surfacing. The device reads to 0.01 in. and has been shown to be accurate to approximately 1.3 mm (0.05 in.). An example of subgrade and surface profile elevations taken at a typical site is shown in Figure 5.

FIGURE 3 Typical site.

FIGURE 4 Pore pressure probe, two views.

Vehicles and Traffic

All vehicles used for traffic were equipped with CTI systems which allow the changing of the tire pressure for each wheel while en route. For the loaded trucks, CTI tire pressures were 482 kPa (70 psi) for the steering tires and 358 kPa (52 psi) for the driver and trailer wheels. For the unloaded logging truck and the dump truck (used to simulate an unloaded log truck), the tire pressures were 482 kPa (70 psi) for the steering and 207 kPa (30 psi) for the driver wheels. For sites where highway tire pressure was designated, all tires were set to 620 kPa (90 psi).

The loaded trucks traveled downhill, and the unloaded trucks traveled uphill. Specific sections were designated to have highway tire pressures, whereas others had only CTI low tire pressures. The total traffic in-

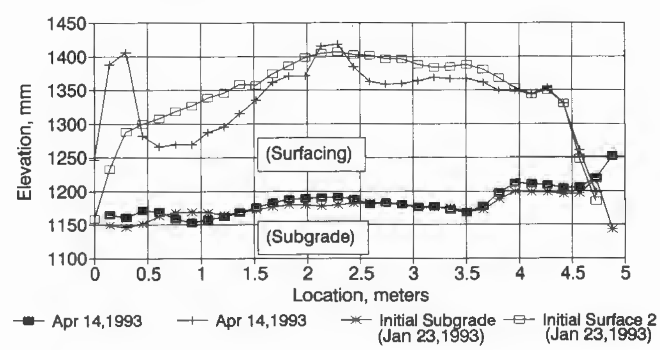


FIGURE 5 Typical subgrade and surface profiles, Site 1993-9.

cluded 1,025 loaded truck laps in 1992 and 787 laps in 1993, with an approximately equal number of unloaded truck laps.

Summary of Test Results

Surface Rutting

The progression of rutting for each site was plotted. Some of these sites are plotted as a function of date in Figures 6 and 7. Site 1992-7 is plotted in Figure 8 as a function of equivalent drive axle repetitions. Table 1 gives the maximum ruts for both surfacing and subgrade and the change in thickness from initial. As seen in Figures 6 and 7 and Table 1, Sites 1 and 8 had the least amount of surfacing thickness and the greatest amount of surface rutting in 1992. In 1993 Sites 1, 2, 9, and 10 had the greatest amount of surface rutting, again with the least aggregate thickness. All these sites had an initial thickness of less than 178 mm (7 in.).

Sites 1992-1 and 1992-8 and 1993-9 and 1993-10 experienced drainage problems during periods of heavy rainfall that were not typical of the other sites. Site 1993-9 is in the same location as Site 1992-8. These sites exhibited high surface rutting after these rainy periods, whereas only Site 1992-8 had any significant subgrade rutting.

As previously stated, Sites 1993-1, 2, 5, and 9 had an open-graded aggregate that was specified to be of good quality. These sites also exhibited the greatest amount of surface rutting in 1993, which can be attributed to the low surfacing CBRs caused by the gradation of this aggregate (the aggregate CBRs ranged from 18 to 28).

Subgrade Rutting

In this study, subgrade rutting, which is typically less than 7.6 mm (0.3 in.), was considered insignificant

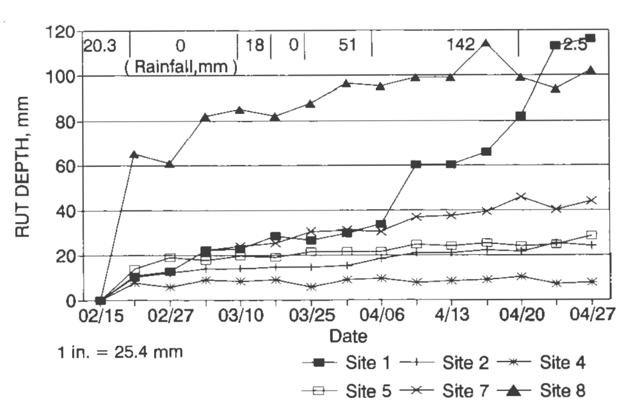


FIGURE 6 Surfacing ruts, 1992.

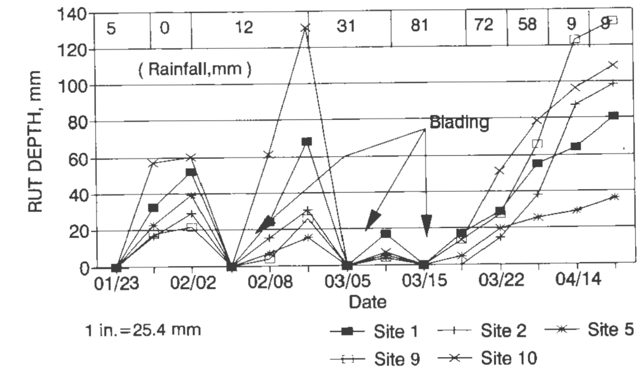


FIGURE 7 Surfacing ruts, 1993.

when compared with surface rutting. Because of drainage problems, low subgrade CBR, and low initial thickness, Site 1992-8 was the exception. Subgrade material pumped through the surfacing aggregate of this site. The surfacing thickness changed from an original 132 mm (5.2 in.) to 58 mm (2.3 in.) after traffic. When the surfacing thickness was reduced to a critical value, subgrade deformation accelerated.

Aggregate and Subgrade Strength Variations

CBR and DCP tests were measured before, during, and after traffic and tended to fluctuate with the weather and with density changes due to the traffic. The surfacing and subgrade field CBR values given in Table 1 were used along with values correlated to DCP testing.

In addition to the initial field CBRs, data analysis was performed using CBRs correlated from the DCP values. The DCP CBRs for 1992 were averaged from all DCP values obtained that year because there was relatively little rainfall in 1992, and rutting progressed

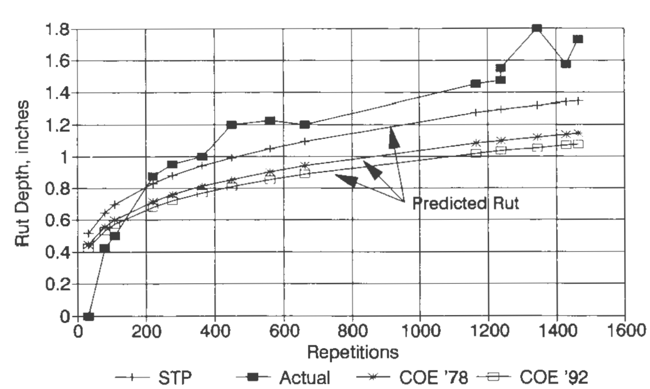


FIGURE 8 Site 1992-7: DCP CBRs C1 = 32.7, C2 = 6.4.

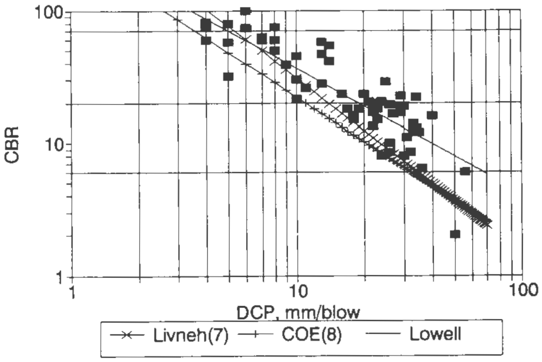


FIGURE 9 Field CBR versus DCP.

slowly. In 1993 the first set of DCP values obtained was used in further analysis. This year had considerably more rainfall, with significantly changing material properties.

Environmental Factors

Rainfall during the testing period ranged up to 52.6 mm (2.07 in.) per day, with one 12-day period accumulating 142 mm (5.6 in.) when traffic was being applied. The longest period involving traffic without rain was 9 days. Typically, because of low temperatures, tree canopy, and high humidity, the road materials stayed damp for a considerable period after a substantial rainfall.

Analysis of Test Results

Relationship Between CBR and DCP

For this project, 77 field CBRs and 115 DCP readings were taken. The tests of the DCP reading compared with field CBR that were taken at the same site at approximately the same time are plotted in Figure 9. Also plotted are two relationships for converting DCP readings to CBR values, those of Livneh (*7*) [$CBR = 646/(DCP)^{1.32}$] and the Corps of Engineers (*8*) [$CBR = 292/(DCP)^{1.12}$].

As seen in Figure 8, the data obtained from this study indicate a closer match between field CBRs and the Livneh correlation, which tends to give higher CBR values for the same penetration than does the Corps of Engineers equation.

For the Lowell test data, a regression analysis was done using the 77 field CBRs with the correlated DCP values. This resulted in the following relationship:

$$CBR = \frac{320}{DCP^{.943}} \qquad R^2 = .76$$

Surface Rutting, Aggregate Thickness, and Thickness Loss

The relationships between surface rutting, traffic (date), maintenance, and rainfall are shown in Figures 6 and 7. The most significant rutting occurred during periods of higher rainfall. At Site 1992-8, the largest increment of rutting, 66 mm (2.6 in.), occurred during the first two traffic periods when 20 mm (0.8 in.) of rain was recorded. Sites 1992-1, 7, and 8 developed rut increments of 48, 15, and 18 mm (1.9, 0.6, and 0.7 in.), respectively, during the traffic period of April 6 through April 20, when 142 mm (5.6 in.) of rain was recorded.

The surfacing thickness of each site was monitored from initial construction through the testing periods in the same manner as the rutting. The loss of thickness can be attributed to densification, local shear of aggregate, and aggregate loss.

After 1,025 or less loaded log truck had passed, the reduction in surfacing thickness at the sites during the testing ranged from 5 to 74 mm (0.2 to 2.9 in.), measured from the wheel path to the subgrade-surface interface. In 1992 the sites with good-quality aggregate and subgrade CBRs greater than 11 resulted in a thickness loss of less than 13 mm (0.5 in.). Sites 1992-1 and 8, with thicknesses of less than 140 mm (5.5 in.), experienced a maximum thickness loss of 74 mm (2.9 in.). These sites also had drainage deficiencies during periods of heavy rainfall.

Aggregate densification was monitored periodically by performing density tests. Densification ranged from 32 to 176 kg/m^3 (2 to 11 lb/ft^3), which accounted for approximately 7.6 mm (0.3 in.) of thickness loss. At several sites, aggregate thickness increases were measured outside the wheel path, indicating lateral displacement of the aggregate.

Typically, the loss of surface thickness was 50 percent or more of the surfacing rut value. Since the subgrade rut was typically less than 15 percent of the surfacing rut, the surfacing rut appears to be more directly affected by lateral movement of the surfacing, resulting in changes in surface thickness, than it is to subgrade rutting.

Subgrade Rutting

The total subgrade rut for any of the sites did not exceed 10 mm (0.4 in.) except at Site 1992-8 (designated Site 1993-9 the following year), and the scarified subgrade Site 1993-5. Fourteen of the 18 sites tested had total subgrade ruts of less than 7.6 mm (0.3 in.). When the sites with subgrade rutting greater than 7.6 mm (excluding Site 1992-8 due to drainage deficiencies) are considered, the subgrade rut was about 10 percent of the surface rut for Sites 1992-1 and 1993-9 and 40 percent for Site 1993-5. For the sites with less than 7.6 mm subgrade rutting, the subgrade rut averaged 14 percent of the surface rut.

Maintenance Implications

The only sites with significant subgrade rutting were sites that, because of drainage deficiencies from lack of side ditches, had a detrimental aggregate thickness loss. This thickness decreased to a critical value, which allowed the subgrade to become overstressed. At the other sites, the surface rutting was not primarily due to subgrade rutting and could be treated by blading maintenance.

As shown in Figure 7, the rut progression after maintenance could be equal to, greater than, or less than the initial rut progression. These sites had DCP values indicating CBR ranges of 10 to 27 for the surfacing and 5 to 8 for the subgrade. These are relatively low; however, the rut progression was generally less than or equal to the initial rut progression even though the aggregate surfacing was not compacted with a roller after blading.

Effect of Aggregate Quality and Moisture Conditions on Rutting

With the exception of Site 1993-10, the poor-quality aggregate sites (1992-5 and 6; 1993-6, 7, and 10) resulted in a very low amount of surfacing rutting, less than 28 mm (1.1 in.), and a minimal amount of subgrade rutting, less than 7.6 mm (0.3 in.). These sites typically had high surfacing CBRs (32.5 to 74), high subgrade CBRs (13.5 to 22.5) during dry conditions, good drainage, and an average of 203 mm (8 in.) of surfacing material. These factors, coupled with relatively dry conditions in 1992, account for the low rutting at these sites.

Higher rainfall levels in 1993 suggested that a difference in rutting between the good and poorer-quality aggregate sites would occur. However, the new sites that were specified for good-quality aggregate were low-strength (CBR) sites because of the poor aggregate gradations.

The 1993 aggregates with poor-quality tests and poor gradations were susceptible to strength decreases during rainfall extremes. The field and DCP CBR tests varied widely for these sites.

Effect of Tire Pressure on Rutting

In 1992 two of the eight sites (7 and 8) had CTI traffic; in 1993 three of the ten sites (8, 9, and 10) had CTI traffic. Sites 1992-8 and 1993-10 had drainage problems and are not good sites from which to draw conclusions. The CTI sites were constructed on weaker soils and less aggregate thickness in an effort to test CTI under extreme conditions. For these reasons, it is not possible to draw definitive conclusions regarding the performance of the CTI compared with highway tire pressures. However, some observations were noted:

1. During 1992 the highway tire pressure sites typically performed as predicted by STP or slightly better, as indicated by the R^2 values in Table 1 and additional analysis provided on the Lowell test road (*9*). The CTI site (1992-7 only) performed close to the STP predictions. This would support the STP method of addressing traffic and tire pressure, possibly being slightly optimistic toward CTI.
2. During 1993 the highway tire pressure sites typically performed better than had been predicted by STP. The CTI sites generally performed as predicted by STP; some sites performed better. This indicates that STP handles traffic and tire pressure acceptably and is slightly optimistic regarding CTI.
3. An analysis of the effect of tire pressure on individual sites showed a dramatic increase in rutting of the 1993 CTI sites 8 and 9 after the traffic was changed from CTI to highway tire pressure shortly after the March 26 measurement. This indicates large benefits for CTI over highway pressures, especially when sections are minimally designed.

In the Lowell rutting study, the lack of a large difference between highway and CTI rutting can be partially explained by the tire pressures used. For the standard highway situation, 620 kPa (90 psi) was used on all tires. For CTI, the steering tire pressure was reduced to 482 kPa (70 psi). In reducing the steering tires from 620 to 482 kPa (90 to 70 psi), the average tire contact pressure decreased from 469 kPa (68 psi) to 427 kPa (62 psi), as determined from a measured load and tire contact prints. In changing from 620 to 358 kPa (90 to 52 psi), the average contact pressure of the driver and trailer tires decreased from 365 to 303 kPa (53 to 44 psi), whereas the contact area radius increased only 11 mm from 128 mm (0.45 in. from 5.05 in.).

Comparison of Rutting with Design Model

Corps of Engineers

The 1978 Corps of Engineers design model (*5*) was used to predict the amount of rutting that would occur at each of the test sites for 1992 and 1993. The actual rutting that did occur was then compared with the predicted rutting by use of linear regression analysis. From this, the R^2-value and other statistics were used to compare different aspects of predicted versus actual rut development.

In 1992, 114 total data points were collected from the test sites. For 1993 the sites were graded before a number of data pairs were obtained. Because of this, only 21 data points were used for this analysis. For these data, the relationship between predicted (PR) and actual (RD) ruts for 135 data points using field CBRs in the prediction model is $PR = .21RD + .40$ with an $R^2 = .39$. In all conditions evaluated, as the actual rut depth increased, the predicted rut generally tended to approach a factor of from 0.2 to 0.7 of the RD value. In this analysis the field CBRs do not appear to contribute to significantly better correlation between predicted rut and actual rut than do the DCP CBRs.

In looking at the R^2-values for the individual 1992 sites, there is a high correlation (greater than 0.5) between predicted and actual ruts for all sites except 1992-3 and 1992-4. These two sites also had the greatest surfacing thicknesses, 234 and 269 mm (9.2 and 10.6 in.), and relatively high subgrade CBRs (11 and 19.5). At both of these sites, the rut prediction equations predicted a higher rut than actually occurred.

Surfacing Thickness Program (STP)

In preparing the STP program, to simplify use of the Corps of Engineers equation, a number of factors and input variables were standardized. This is explained in more detail by Whitcomb et al. (*1*). To analyze the effect this had on rut prediction, rut depth was predicted using both methods before any maintenance activities.

Considering the maximum rut predicted at each of these sites, the STP model predicted a higher rut than the Corps of Engineers equation did. The differences ranged from 1.5 to 16 mm (0.06 to 0.64 in.), or 11 percent to 44 percent, greater when compared with the Corps of Engineers prediction. The difference was typically around 20 percent. The sites with the greatest differences generally occurred where CTI tire pressures were used and the surfacing thicknesses were less than 175 mm (6.9 in.).

In comparing the predicted rut with the actual maximum rut exhibited, the STP model tended to underpredict the rut by an average of 16 percent, whereas the Corps of Engineers model tended to underpredict by an average of 29 percent.

Although the predicted ruts for each site both exceeded and fell short of actual ruts at different sites, the same general tendency for underprediction of ruts was verified by the linear regression analysis comparing actual and predicted ruts. In the equations generated relating predicted rut to actual rut, a coefficient typically less than .7 would need to be multiplied by the actual rut depth followed by the addition of a small constant for equality to be obtained.

Conclusions and Recommendations

Conclusions

The major conclusions and findings of this study are discussed in the following paragraphs.

The STP accurately predicts aggregate surfacing rutting. STP tended to underpredict rut by an average of 16 percent.

Surface rutting is primarily due to densification and lateral aggregate displacement; only a small portion of the rut (typically less than 20 percent) is observed in the subgrade. Subgrade rutting can be significant if aggregate thicknesses are at a minimum level to initiation of subgrade shearing.

The DCP is a useful tool for evaluating material strength properties. A good correlation exists between CBR and DCP—it is a rapid test that supports the ability to perform a large number of tests.

The CTI showed improved performance over highway tire pressure sites when used at the same sites later in the testing. The beneficial effect of CTI may not be as much as predicted by STP. However, this may be due to the lower difference between CTI and highway tire pressures used in this study than was used in previous efforts.

Not surprisingly, low compaction and poor drainage contribute to significantly more rutting than would normally be experienced.

Recommendations

For future studies of this type, it is recommended that a more significant difference between actual tire contact pressures for highway and off-highway conditions be obtained than was used for this study.

Additionally, the pressure probe used to measure elevations in this study worked very well by providing accurate measurements in a digital form that could easily be reduced and analyzed with a personal computer.

ACKNOWLEDGMENTS

This project was carried out under the sponsorship of the FHWA CTIP program administered by the San Dimas Equipment Development Center in cooperation with additional research being performed by the Intermountain Research Station of the USDA Forest Service.

Gratitude is expressed to the Lowell Ranger District for their cooperation in implementing the project. Thanks are also given to Foglio Trucking for their assistance in providing a vehicle, driver, and record keeping.

REFERENCES

1. Whitcomb, W. G., M. T. Yapp, and M. Myers. *Aggregate Surfacing Design Guide*. Final Report. USDA Forest Service, Portland, Oreg., 1990.
2. Grau, R. W., and L. B. Dell-Moretta. Effects of Variable Tire Pressure on Road Surfacings. In *Transportation Research Record 1291*, TRB, National Research Council, Washington, D.C., 1991, pp. 313–328.
3. Smith, D. M. *Effects of Variable Tire Pressure on Road Surfacings*, Volume II: *Analysis of Test Results*. Geotechnical Laboratory, U.S. Army Corp of Engineer Waterways Experiment Station, Vicksburg, Miss., 1992.
4. Yapp, M. T. Y., J. Steward, and W. G. Whitcomb. Existing Methods for the Structural Design of Aggregate Road Surfaces on Forest Roads. In *Transportation Research Record 1291*, Vol. 2, TRB, National Research Council, Washington, D.C., 1991, pp. 41–57.
5. Barber, V. C., E. C. Odom, and R. W. Patrick. *The Deterioration and Reliability of Pavements*. Technical Report S-78-8. U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss. July 1978.
6. Sacco, D. A., R. L. Copstead, and D. J. Janssen. Liquid Level Gauge for Measuring the Cross-Sectional Deformation of Aggregate Surfaced Roadways. In *Transportation Research Record 1426*, TRB, National Research Council, Washington, D.C., 1993, pp. 74–80.
7. Livneh, M. Validation of Correlations Between a Number of Penetration Tests and In-situ CBR Tests. Presented at the 68th Annual Meeting, TRB, National Research Council, Washington, D.C., Jan. 22–26, 1989.
8. Webster, S. L., R. H. Grau, and T. P. Williams. *Description and Application of Dual Mass Dynamic Cone Penetrometer*. Project AT40-RC-003. U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., May 1991.
9. Truebe, M., and G. Evans. *Lowell Surfacing Thickness Design Test Road*. EM-7170-15. USDA Forest Service, Washington, D.C., Sept. 1994.

Guidelines for Structural Design of Low-Volume Rural Roads in Southern Africa

G.D. van Zyl, *Council for Scientific and Industrial Research, South Africa*
A.T. Visser, *University of Pretoria, South Africa*
J.A. du Plessis, *Department of Transport, South Africa*

An extensive project was recently completed in South Africa under the auspices of the Department of Transport to develop guidelines on the standards for roads that carry up to 400 vehicles per day. These guidelines were developed because traditional standards could not be justified. The guidelines encompass all aspects of road design, construction, and maintenance. The aim of this paper is to present the structural designs and material quality guidelines that were developed. In the structural design, special attention is given to evaluating the existing unpaved road since the economic construction costs do not provide for major realignment. The in situ conditions, monitored with a dynamic cone penetrometer, provide the input to the structural design, which is in a catalog format. A catalog of pavement thickness designs was developed using sophisticated analysis techniques, such as elastoplastic modeling, to allow the use of materials that fall outside the traditional specifications. The main emphasis is on using the existing road without disturbing the traffic compaction that was applied over many years and on adding the minimum amount of material. Of special interest is the approach that was adopted for selecting the asphalt surfacing, which is based on expected performance and maintenance and life-cycle costs. This paper contains the state of the art in low-volume road pavement design in South Africa and these guidelines are considered a major step in economically extending quality service to the sparsely populated rural areas. The guidelines should also be valuable to practitioners worldwide.

More than 70 percent of the road network in South Africa is still unpaved. The majority of these roads carry fewer than 400 vehicles per day (vpd). Using traditional standards, upgrading these roads would be warranted only if the traffic volumes exceed about 400 vehicles per day. The traditional approach has led to the perception of a huge gap between the standards of gravel or earth road facilities and surfaced road facilities.

In 1992 the Department of Transport in South Africa initiated a comprehensive research project to develop guidelines on standards for roads that carry up to 400 vpd for which upgrading is warranted (*1*). These guidelines are based on local research, performance of light pavement structures, and practical experience. They further encompass all aspects of road design, construction, and maintenance.

The purpose of this paper is to present the structural designs and material quality guidelines that were developed. The paper first discusses the design philosophy. Thereafter, the design catalog and application to existing gravel roads are presented. The choice of surfacing is briefly discussed and finally the applicability and conclusion and recommendations are presented.

Design Philosophy

The upgrading referred to here is primarily the provision of a bituminous surfacing to keep water out of the pavement structure, to protect the underlying layers from the disruptive effects of traffic, and to provide an all-weather, dust-free riding surface. The roads in question are all considered to be short local-access roads; paved links shorter than the existing main routes should not be created because an unplanned through route would result.

The unpaved roads in the region vary from tracks to well-designed and -constructed gravel roads, resulting in a large variation in serviceability. In many cases, these roads have been in place for long periods, and problem areas have already been identified and improved. Through frequent regraveling, drainage improvements, and traffic compaction, some of these roads already have sufficient structural capacity to carry the expected traffic for more than 10 years, provided that a bituminous surface is applied.

The upgrading approach consists of the following steps, which will be discussed below:

- Determine the pavement strength required to ensure good performance and therefore low maintenance during the design period;
- Test the existing gravel road structure and determine what type of strengthening, if any, is required before surfacing; and
- Select an appropriate surfacing for the maintenance capability and environment.

Determining Required Pavement Strength

Design Strategy

The economic analysis period is a realistic cost period, which is used to compare the cost of upgrading options. The analysis period recommended for low-volume roads is dependent on the existing geometrical standards and the likelihood for upgrading needed in this regard. A 20-year period is recommended where the alignment is fixed and 10 to 15 years where uncertainty exists.

Experience has shown that appropriate and properly constructed bituminous surfacings can be expected to last for 10 years. The cost of these surfacings can exceed 50 percent of the cost of upgrading; the pavement should therefore be able to carry the load for at least 10 years. However, in cases where it will be difficult or impractical to carry out structural rehabilitation, for example, in difficult terrain or because of financial constraints, a longer period of 15 to 20 years can be selected. The cost differential between 10 and 20 years can sometimes be surprisingly low and the consequences should be analyzed.

Design Traffic

Traffic Loading

For the structural design process, an estimate of the traffic loading (expressed as cumulative equivalent 80-kN axle loads over the structural design period) is required. At its simplest, this is derived from the average daily traffic in both directions and the percentage of heavy vehicles. Since heavy vehicles (trucks and buses) weigh so much more than cars, for all practical purposes it is sufficient to consider the loading from the heavy vehicles alone and ignore the cars. In addition, the growth rate over the structural design period is estimated. The simplest form of estimating the loading of heavy vehicles is to use tabulated values representing average conditions. A rough estimate is made of the type of heavy traffic, and the average number of equivalent 80-kN axles (E80s) per heavy vehicle is than read from a table such as Table 1 or Table 2. This factor is then multiplied by the number of heavy vehicles in both directions to obtain the average daily E80s.

Tables 1 and 2 reflect typical loads in South Africa. Because of the variation in the legal axle load limit and

TABLE 1 Average E80s per Heavy Vehicle (2)

LOADING OF HEAVY VEHICLES	E80/HEAVY VEHICLE
Mostly unladen	0,6
50 % of heavy vehicles laden and 50 % unladen	1,2
> 70 % of the heavy vehicles fully laden	2,0

TABLE 2 Average E80s for Different Heavy-Vehicle Configurations (2)

Vehicle type	Average E80s per vehicle	Range in average E80s per vehicle found at different sites
2-axle truck	0,70	0,30 - 1,10
2-axle bus*	0,73	0,41 - 1,52
3-axle truck	1,70	0,80 - 2,60
4-axle truck	1,80	0,80 - 3,00
5-axle truck	2,20	1,00 - 3,00
6-axle truck	3,50	1,60 - 5,20
7-axle truck	4,40	3,80 - 5,00

Note: * E80s of a fully laden 2-axle bus = 2,77

levels of control of overloading, these values might be unsuitable for other countries in the region.

E80 Growth Rate

The growth rate of heavy vehicle loading (E80s) can be different from the growth rate of heavy vehicles. This can be due to the growth rate of the number of axles per vehicle and the extent to which the vehicles are loaded on average. An increase in the permissible axle load will also lead to an increase in E80 growth rate. The E80 growth rate will further depend on whether the facility is used for tourism, farming, or industrialization and expected future developments. Therefore, where possible, the growth rate should be based on specific information. Generally the E80 growth rate is expected to range between 2 and 10 percent per annum.

Design Traffic

The design traffic (in terms of E80s) is calculated from the number of E80s at the start of the analysis period, the growth rate, and the structural analysis period. Care should be taken to allow for attracted traffic once the existing gravel road is surfaced.

Distribution per Lane

Low-volume roads (LVRs) typically consist of two lanes, one lane per direction. If the road has only one lane per direction, the cumulative E80s per direction constitute the design traffic. In some cases one lane may be carrying loaded vehicles, whereas the other is carrying empty vehicles. A distribution factor between lanes larger than 0.5 should be considered for the critical lane in such cases. If the road consists of only one lane carrying traffic in both directions, the cumulative E80s in both directions should be used.

Sensitivity of Traffic Class to Growth Rate, Loading, and Other Factors

The estimation of design E80s for LVRs should never be calculated as one number but should reflect a range of possible E80s. Therefore, an important design step is to conduct a sensitivity analysis. This will consider variations in factors, such as the E80 growth rate, E80 per vehicle or E80 per axle, initial E80 per day, and structural design period. Certain factors may be more uncertain or may have a larger influence than others for a specific design. By analyzing minimum and maximum scenarios, a range in design E80s can be obtained. The possible range in design E80s should be matched with the E80 ranges in Table 3 and the appropriate traffic class selected.

Pavement Structure

Material Classification

An understanding of the standard classification of road building materials in South Africa is essential to the following discussion. A description of codes, material types, and abbreviated specifications is given in Table 4.

Pavement Materials

The selection of materials for the pavement structure is based on a combination of structural requirements, availability, economic factors, and previous experience. These factors need to be evaluated during the design phase to select the materials that are most appropriate for the prevailing conditions. The selection criteria for materials for low-volume roads are similar to those for high-volume roads. Therefore, certain aspects must be satisfied with regard to the selection of materials:

TABLE 3 Catalog of Pavement Structures

TRAFFIC CLASS	TRAFFIC (E80's)	PROPOSED PAVEMENT STRUCTURES #					
		GRANULAR/GRANULAR		GRANULAR/ CEMENTED	CEMENTED/G RANULAR	CEMENTED/ CEMENTED	ASPHALT SURFACING/ GRANULAR
		DRY/ MODERATE	WET				
E0-1	< 5000	150 G6* 150 G8 150 G9 G10**	150 G5 150 G7 150 G9 G10	150 G5 125 C4 G10	100 C4*** 150 G9 G10	-	25 A^{+} 150 G6 G10
E0-2	5 000 - 30 000	150 G5 150 G7 150 G9 G10	150 G4 150 G6 150 G8 G10	-	100 C4 150 G7 G10	-	25 A 150 G6 150 G7 G10
E0-3	30 000 100 000	150 G4 150 G6 150 G8 G10	150 G4 150 G5 150 G6 150 G7 G10	150 G4 125 C4 150 G7 G10	125 C4 150 G5 G10	100 C4$^{\phi}$ 100 C4 G10	25 A 150 G5 150 G9 G10
E0-4	100 000 - 200 000	150 G4 150 G5 150 G8 G10	150 G3 150 G6 150 G9 G10	150 G4 125 C4 150 G7 150 G9 G10	125 C4 150 G5 150 G7 G10	-	25 A 150 G4 150 G9 G10
E1-1	200 000 - 400 000	150 G4 150 G5 150 G7 150 G9 G10	150 G3 150 G6 150 G8 G10	125 G2 125 C4 150 G9 G10	125 C4 150 G4 150 G7 G10	100 C4$^{\phi}$ 100 C4 150 G7 150 G9 G10	25 A 150 G4 150 G8 G10
E1-2	400 000 - 800 000	125 G2 150 G6 150 G9 G10	125 G2 150 G5 150 G9 G10	150 G2 125 C4 150 G9 G10	$	125 C4 125 C4 150 G7 150 G9 G10	25 A 150 G4 150 G5 150 G8 G10

Double surface treatment assumed on all pavement structures unless otherwise indicated.

* Notation - 150 mm layer of G6 quality material. Layers are designated from top to bottom, with the lower being the roadbed material.

** **Pavement assumed to be supported by in-situ material having a CBR of not less than 3 (G10) and semi-infinite depth.**

*** C4 - cementation of G5, G6 material.

\+ 25 mm asphalt

ϕ Can be combined into one layer of 200 mm thickness.

$ At present, reliable calculations of life expectancy cannot be made for this type of pavement structure.

TABLE 4 Summary of Material Classification (*3*)

CODE	MATERIAL	ABBREVIATED SPECIFICATIONS
G1	Graded crushed stone	Dense-graded unweathered crushed stone: max.size 37,5 mm 86-88 % of apparent density; fines PI < 4
G2	Graded crushed stone	Dense-graded unweathered crushed stone: max. size 37,5 mm 100-102 % mod. AASHTO; fines PI< 6.
G3	Graded crushed stone	Dense-graded stone + soil binder: max size 37,5 Minimum 98 % mod. AASHTO; fines PI < 6
G4	Natural gravel	CBR > 80 ; PI < 6
G5	Natural gravel	CBR > 45 ; PI < 10 ; max. size 63 mm
G6	Natural gravel	CBR > 25 ; max. size < 0,67 layer thickness
G7	Gravel-soil	CBR > 15 ; max. size < 0,67 layer thickness
G8	Gravel-soil	CBR > 10 ; at in-situ density
G9	Gravel-soil	CBR > 7 ; at in-situ density
G10	Gravel-soil	CBR > 3 ; at in-situ density
C3	Cemented natural gravel	UCS 1,5 to 3,0 MPa at 100% mod. AASHTO; max. size 63 mm
C4	Cemented natural gravel	UCS 0,75 to 1,5 MPa at 100% mod. AASHTO; max. size 63 mm

Note: All CBR values referred to in Table 4 are soaked CBRs.

- Adequate bearing capacity under any individual applied load;
- Adequate bearing capacity to resist progressive failure under repeated individual loads;
- Ability to retain that bearing capacity with time (durability); and
- Ability to retain bearing capacity under various environmental influences (which relates to material moisture content and in turn to the climate, drainage, and moisture regime).

Standards should be relaxed only in light of the relevant maintenance capabilities available in the area. If potholes or cracks occur and are not repaired speedily, water ingress could lead to substantial failures. Thus, if the local maintenance capability is poor, it is recommended that less moisture-sensitive materials, or the best material locally available, should be used. In areas with a very high maintenance capability, relaxation of traditional standards may be considered.

Relaxation of Atterberg limits is permitted for low-volume roads in drier climates, provided the material meets the appropriate bearing strength and durability requirements. No relaxation is permitted in wet environments unless the soaked California bearing ratio (CBR) exceeds the specified limits by at least 10 percent, the materials have low moisture sensitivity, and the pavement is well drained. Relaxation of the plasticity index (PI) up to 15 percent for calcretes and ferricretes is permitted.

The normal field compaction requirements for untreated layered materials apply. It is emphasized that the higher the density obtained, the stronger the compacted material will be and the lower the potential rut formation due to densification in service.

If no suitable materials are available locally for base or subbase layers, modification or stabilization with lime, cement, lime slag, or other pozzolanic stabilizers or combinations may be used to improve local materials. Tests such as the initial consumption of lime may show the need to increase the stabilizer content. However, there are both economic and engineering limits to the amount of stabilizer that should be added. For practical purposes, this limit is about 4 to 5 percent. Due to practical constraints with "on the road" mixing in of the stabilizer, minimum limits are usually set at 2 to 2.5 percent of stabilizer. In many cases, this can still result in very high layer strengths (exceeding that which is required); therefore, even if carbonation takes place, sufficient strength exists in the layer.

Pavement Structure

The catalog presented in Table 3 provides a range of structures appropriate to carry the relevant design E80s but does not exclude other possible pavement structures. This catalog was developed using characteristics of typical local materials, field testing and evaluation of existing light pavement structures, and elastoplastic modeling techniques and is described by Wolff et al. in a companion paper in these proceedings. Different pavement compositions consisting of granular layers, cemented layers, or combinations thereof are presented. Of specific importance is the variation of recommended granular pavement compositions in wet or dry to moderate environments. Due to the current unavailability of appropriate transfer functions for design purposes, pavement compositions with bitumen emulsion–treated layers have been excluded.

The most appropriate structure should be selected from an economic analysis based on conditions specific to the project. These conditions normally include aspects such as material availability, maintenance capabilities, construction skills, and established procedures.

Incorporating Existing Pavement in Design

Testing In Situ Strength of Existing Gravel Road

The existing gravel road strength should be used when the road is upgraded to a paved road. This is done by using the existing pavement as part of the new pavement and classifying the existing pavement material strengths in terms of the standard G1 (crushed stone) to G10 (subgrade soil) granular material classification as presented in Table 4. For example, if an existing low-volume gravel road had a wearing course of G4 material and a subbase of G5 material, those layers could be used as the base course and subbase of the paved road, creating a structure that is probably stronger than the catalog specifications. In this example, even if the catalog called for a new pavement of a G5 base course, a G8 subbase, and a G10 subgrade, it would be pointless to place those layers on top of the existing G4 and G5 material. There are parts of South Africa where the in situ subgrade is strong enough to be classified G5, and the bitumen surfacing could be laid on top of this without further layering (although attention would obviously be needed to levels, evenness, and drainage).

To use the existing gravel road strength, the materials in the pavement layers need to be tested for their actual bearing capacity using a dynamic cone penetrometer (DCP) (*4*), and their actual and theoretical bearing capacities need to be compared. For example, a G5 material may have been laid without proper compaction and will only perform as a G6 material. Alternatively, a G6 material may have been so well compacted by traffic over time that it can perform as a G5 material.

The materials in the catalog are classified by their soaked bearing strength (see companion paper by Wolff et al. in these proceedings), and the existing pavement materials need to be classified in terms of their soaked CBR to be related to the catalog. However, the CBR of a material in the field at different moisture contents and densities can vary significantly from its soaked CBR; in general, the drier it is, the higher the field CBR (*5*). Therefore, the DCP-CBR needs to be adjusted to the equivalent soaked CBR.

The preferred method for determining the soaked CBR of the existing gravel road materials is to take many samples and test them in the laboratory. At the same time, field density tests of all layers should be performed to ensure that their compaction is adequate. This can involve considerable testing and cost. However, a simpler, although less accurate, method is to use the DCP for most of the testing in conjunction with a limited number of laboratory soaked CBR tests. Then the design can be based on soaked CBRs estimated from the relationships between field DCP-CBR and soaked CBR (Table 5 for roads that are presently gravel) and cross-checked with the laboratory CBRs.

The compaction can be checked also. If the field DCP-CBRs estimated from the laboratory soaked CBR results are less than those actually found in the field, the existing gravel road has been well compacted (by traffic) and is suitable for incorporation in the design. If, however, the actual field DCP-CBRs are less than those from the laboratory, compaction is lacking and the existing gravel layer should be ripped and recompacted. Alternatively, compaction can be checked if sufficient field density tests have been performed, and the results compared to specified Mod. AASHTO densities.

Procedure for Using Existing Gravel Roads in Design

To ensure the cost-effective design of low-volume pavements, the following simple procedure is provided to optimize the in situ strength of gravel roads.

Step 1: Calculate Design Traffic and Select Traffic Class

Using the guidance above, a range of possible design E80s is obtained. By matching this range with the ranges given in Table 3 (second column), a suitable traffic class can be selected.

Step 2: Complete Tests Along the Road

DCP testing is performed along the length of road. The frequency of tests should generally follow the recommendations below, but a visual inspection may indicate adjustments to the frequency. If the road is uniform, the frequency can be reduced; if it is variable, the frequency should be increased. The basic frequency should be

- Test at the rate of five DCP tests per kilometer, with the tests staggered as outer wheel track–inner wheel track one side, outer wheel track–inner wheel track other side, center line, etc.;

TABLE 5 Approximate Relationship Between Soaked CBR and Field DCP-CBR for Gravel Road

Material classification	Soaked CBR	APPROXIMATE FIELD DCP-CBR : GRAVEL ROAD					
		Subgrade		Wearing course			
		wet climate	dry climate	very dry state	dry state	moderate state	damp state
G4	80			318	228	164	117
G5	45			244	175	126	90
G6	25	59	65	186	134	96	69
G7	15	45	50	147	106	76	54
G8	10	38	43				
G9	7	33	37				
G10	3	20	24				

Notes 1 The inter-relationship between soaked CBR and field DCP-CBR is approximate due to the variability of moisture contents, materials, test methods, and densities. It assumes that the density relates approximately to the field density expected for that layer. More research will give more confidence to this relationship.

2 The moisture contents that this table are based on are estimated moisture contents, based on various field studies and experience; they can vary in practice from the values assumed here. For the wearing course they are (expressed as the ratio of field moisture content to Mod AASHTO optimum moisture content): very dry state = 0,25; dry = 0,5; moderate = 0,75; damp = 1,0.

3 This table has been developed from Table 22 and equation 36 of Emery (1992) (5)

- Conduct an additional test at each significant location picked up in the visual survey, such as particular failure areas; and
- Ensure that at least eight DCP tests are performed per likely uniform section to provide adequate data for the statistical analysis.

It is recommended that at least two samples per kilometer be taken to check laboratory soaked CBR, Atterberg limits, and the in situ moisture content of each layer.

Step 3: Divide Road Into Uniform Sections for Upgrading

The results of the investigation, including the DCP testing and visual assessment, will enable the division of the length of road into relatively uniform sections for the purposes of upgrading. The minimum length of a section should be 0.1 km, but preferably 1 km. On long lengths of road with uniform conditions, the length of sections may be 10 km. Note that the construction of sections shorter than 0.5 km can be awkward. Low DCP results may occur in a spot that was identified in the visual survey as an isolated problem area; these are typical of an isolated drainage problem. Such section lengths should be repaired individually rather than regarded as being representative.

Step 4: Calculate Representative Layer Strengths for Each Section

The representative DCP layer strengths for each section are deemed to be those values below which only 20 percent of the measured DCP-CBRs lie. The easiest method for calculation is to analyze the field DCP results in uniform layer thicknesses, with each layer being, say, 150 mm thick. The actual rate of penetration is converted to the in situ DCP-CBR for each test (4). The representative DCP-CBR for each layer is then found statistically to provide a safety margin against the variability of material within the section. A normal distribution of data is assumed and Student's *t* distribution at the 80 percent level is used.

Step 5: Convert Layer Strengths to Material Classification

The representative DCP-CBR values must be converted to layer thicknesses and material types (i.e., 150 mm G5). The material type can be estimated from the soaked CBR that is obtained from the field DCP-CBR and moisture content during testing (Table 5 for roads that are presently unpaved) and cross-checked with the laboratory CBRs.

Step 6: Compare In Situ Structure With Required Pavement Structures

The in situ pavement structure (now expressed in terms of layer thickness and material classification) is compared with the catalog design (Table 3) for the relevant traffic class. This will indicate what new layers, if any, are required, and what layers need to be reworked or stabilized to improve their classification.

If additional layers are required, materials that meet the requirements must be located. In the case where suitable materials are available locally, the decision to modify and stabilize local materials or to import materials is made on economic grounds.

Choice of Surfacing Type

Materials

Materials used for surface seals consist mainly of a bituminous binder and aggregate (sand and/or crushed stone). The existing crushing strength requirement of 210 kN 10 percent fines aggregate crushing value (FACT) can be relaxed in light of the adequate performance of materials with lower crushing strength (120 kN 10% FACT) on low-volume roads, provided that construction with a steel-wheeled roller is restricted. [The 10% FACT is the load in kilonewtons required to crush a sample of −13.2 +9.5 mm aggregate so that 10 percent per mass of the total test sample will pass a 2.36-mm sieve (6).]

Polishing stone in the surfacing lowers the low-speed skid resistance of the surfacing under wet conditions and is particularly important when the texture depth of the surfacing is shallow. The rate of polishing, apart from the stone properties, is mainly dependent on the traffic volume. This implies that traffic volumes less than 400 vpd could take 10 times as long to polish stone to the same extent as 4,000 vpd. For low-volume roads, polishing is rarely a problem and the polishing stone value requirement can therefore be reduced for stone used for single and multiple seals and Cape seals.

For low-volume roads, the surfacing is important for good performance; therefore, choice must be made carefully. The choice of appropriate surfacing is based on performance, and then on cost.

The following discussion has been taken largely from South African Bitumen and Tar Association Manual 10 (7).

Performance

The performance of bituminous surfacings is determined by the environment, maintenance capability, and gradient. The restrictions on choice are progressive and sequential. Thus, a restriction in any one aspect is sufficient to limit the choice of surfacing.

Environment

The environment that the road traverses plays a major role in the choice of surfacing. Environment in this case includes climate, surroundings, topography, and institutional capability. From assessments on more than 100 low-volume roads, four different environments were identified:

- **First world, high pavement standards:** Pavements are generally well designed and constructed. In this environment any standard surfacing can perform.
- **First world, lower pavement standards:** This environment typically represents the roads of a small road authority with a restricted budget. Care should be taken in constructing thin surfacings, such as sand seals, thin slurries, and single stone seals.
- **Wet, hilly environments:** The maintenance capability and gradient of the road dictate the performance (see Tables 6 and 7).
- **Third-world environments:** Different stresses and low maintenance often result in loss of the complete investment (see Tables 6 and 7).

Maintenance

The maintenance capability of the road authority has a major effect on the performance of the surfacing. Light seals can give good performance provided they receive

TABLE 6 Choice of Surfacing for Rural Low-Volume Roads by Maintenance Capability

MAINTENANCE CAPABILITY	DEFINITION	SURFACING RECOMMENDATION
High	Can perform any type of maintenance	any
Medium	Routine maintenance, patching and crack sealing on a regular basis. Typically no maintenance management system[b]	asphalt, Cape Seal, slurry[a] double seal, single seal
Low	Patching done irregularly, no committed team, no inspection system	asphalt, Cape Seal, thick slurry, double seal[c]
None	No maintenance	asphalt

Notes a: thin slurries can lead to construction problems

b: it is not essential to have a maintenance management system, but its presence indicates a certain level of capability

c: this is sensitive to construction problems

adequate routine maintenance. Conversely, if there is no maintenance capability, only those surfacings that are inherently tough can survive. Maintenance capability varies widely because the capabilities of the authorities vary. The reasons for the variation include the level of expertise of the road authority, the funds available, security problems (risk, riots, etc.), and the quality of personnel. Lack of maintenance must be considered a part of the stresses on the surfacing, and the appropriate surfacing must be selected to cope.

Gradient

Gradient limits are important to minimize the damage caused by water running along the surfacing (parallel to the center line), as opposed to water running off the surfacing. Water flowing over the bituminous surfacing causes damage on roads with steep gradients, particularly those with curbs, such as are found in hilly areas. There is a maximum water velocity for each type of surfacing above which the surfacing is damaged by stone plucking and scour. Water velocity is related to gradient; therefore, gradient is used to select appropriate surfacings that will be able to resist this type of stress.

Gradient limits also apply to minimizing damage caused by shoving. Shoving occurs when the bituminous surfacing slips across the base course. For this reason, shoving limits are applicable only to an initial seal. It is much less common to find shoving of a reseal; in such cases, either a built-in construction defect exists (e.g., lack of tack coat) or the underlying surfacing is already

TABLE 7 Choice of Surfacing for Rural Low-Volume Roads by Gradient

GRADIENT	SURFACING RECOMMENDATION FOR INITIAL SURFACING
< 6%	any surfacing
6 - 8%	asphalt, Cape Seal, thick slurry[a], double seal[b], single seal[b], sand seal[b]
8 - 12%	asphalt, Cape Seal, double seal[b], single seal[ab], sand seal[ab]
12 - 16%	asphalt, Cape Seal[ab], double seal[ab]
> 16%	concrete block/concrete

Notes: a: not on stabilised base course

b: not if water flow is being channelled by kerbs or berms shoving and the reseal merely compounds the problem. The gradient limit to guard against shoving depends partially on the base course; a rough base course is more resistant to shoving than a smooth one. A stabilized base course is sensitive to shoving, and this sensitivity is accentuated on small-radius curves carrying many heavy vehicles. A base course with a thin layer of fines at the top may lead to shoving.

Intersections

Where the road is subject to turning vehicles (such as mine or industrial entrances and intersections), thin seals are generally not recommended. In general, the heavier the vehicles, the stronger the surfacing should be. The application of a fog spray and a blinding layer of sand or a thin slurry over a stone seal at intersections will reduce stone loss and subsequent potholing. In cases of many heavy vehicles turning, only asphalt concrete, epoxy asphalt, concrete, or concrete blocks are recommended.

Choice of Most Cost-Effective Surfacing

Once the appropriate surfacing types have been chosen from the performance viewpoint, their life-cycle cost should be determined to enable the selection of the most cost-effective and affordable surfacing type.

Conclusion and Recommendations

Road authorities' persistence in using surfaced roads of a high standard has led to a huge gap in quality between unpaved and paved roads, rendering it difficult to economically justify the upgrading of unpaved roads that carry fewer than 400 vpd. Cheaper solutions that still provide acceptable facilities for road users can be provided by changing the philosophy of design so that shorter structural design lives are used in the analysis and by making optimum use of the existing strength of gravel roads.

Procedures to investigate the existing gravel road structure with a dynamic cone penetrometer have been tested over a period of more than 10 years and have resulted in substantial savings to both road authorities and road users.

Investigations into the variability of moisture content in pavement structures (6) improved the understanding and use of the DCP in adjusting the measured in situ CBR to the expected in situ CBR of the pavement layers of a surfaced road, after equilibrium is reached. However, Table 5 will have to be refined as more information becomes available.

Bituminous surfacing seals and thin asphalt surfacings can perform well in southern Africa provided that the environment, maintenance capabilities, gradients, and the actions of traffic are properly taken into account. Understanding the limitations of different surfacing strategies will ensure appropriate, and therefore cost-effective, road networks.

The guidelines presented in this paper are the result of extensive research and practical experience in South Africa over the last decade and should be applicable to other countries in the region and elsewhere in the world where climates and environments are similar.

REFERENCES

1. *Guidelines for Upgrading of Low Volume Roads.* Report RR92/466/2. Department of Transport, Pretoria, South Africa, 1993.
2. *Traffic Loading for Pavement and Rehabilitation Design.* Technical Recommendations for Highways (TRH 16). Committee of State Road Authorities, Pretoria, South Africa, 1991.
3. *Guidelines for Road Construction Materials.* Technical Recommendations for Highways (TRH 14). Committee of State Road Authorities, Pretoria, South Africa, 1991.
4. Kleyn, E. G., and M. J. J. Van Heerden. Using DCP Soundings to Optimise Pavement Rehabilitation. In *Proc., Annual Transportation Conference*, Johannesburg, South Africa, 1983.
5. Emery, S. J. *The Prediction of Moisture Content in Untreated Pavement Layers and an Application to Design in Southern Africa.* CSIR Research Report 644, DRTT Bulletin 20. Council for Scientific and Industrial Research, Pretoria, South Africa, 1992.
6. National Institute for Transport and Road Research. *Standard Methods of Testing Road Construction Materials* (TMH1). Council for Scientific and Industrial Research, Pretoria, South Africa, 1986.
7. *Appropriate Standards for Bituminous Surfacings.* Manual 10. South African Bitumen and Tar Association, Cape Town, South Africa, 1992.

Design Catalog for Low-Volume Roads Developed for South African Conditions

H. Wolff, *Crous Wolff Associates, South Africa*
S.J. Emery, *Isando, South Africa*
G.D. van Zyl and P. Paige-Green, *Council for Scientific and Industrial Research, South Africa*

The development of a design catalog for low-volume roads using granular base, subbase materials, and bituminous surfacings is described. The *S-N* design method was used for the development of the catalog. This is a mechanistic design method that is based on elastoplastic behavior, which implies inelasticity and nonlinearity of the material. It uses the principle of *S-N* curves, where *S* denotes a certain stress level and *N* denotes the number of stress repetitions to a failure condition to consider the accumulation of plastic strain (rutting) with each load cycle. The *S-N* design curves were developed from accurate measurements of elastic and plastic strains in granular layers under heavy vehicle simulator (HVS) testing of numerous pavements incorporating such layers. The method was verified by comparing the carrying capacities of 23 low-volume roads determined from field measurements of traffic and rut with the carrying capacities of the same roads calculated mechanistically with the *S-N* method based on pavement structures obtained through in situ and laboratory testing of materials taken from the roads and classified by the specification normally used for pavement materials in South Africa. Reasonable agreement was found between the carrying capacities determined from field measurements and those determined mechanistically with the *S-N* method. This agreement validated the use of the *S-N* method for the development of the catalog. The approach to the selection of materials for low-volume roads varies from other approaches since little relaxation of the materials classification used for high-volume roads is permitted. The "adjustment" for low-volume roads is made in the design catalog compiled with the *S-N* design method. The design catalog was compared with other catalogs, such as Transport Research Laboratory Road Note 31. It was generally found that pavement structures having fewer selected layers and constructed with lower-quality material are required by the low-volume road catalog described in this paper to get the same performance.

In South Africa the pavement structure is commonly selected from a catalog of suitable designs. The TRH 4 (*1*) catalog is widely used for rural roads and all traffic levels. However, TRH 4 is limited in its treatment of low-volume roads. It caters to the lowest traffic category of less than 200,000 equivalent 80-kN axles (E80s) (approximately 18 kips) per lane over the design life of the road. The next lowest traffic category catered to is 800,000 E80s per lane over the design life of the road. The recommended pavement structures are conservative for traffic volumes of less than 500,000, 100,000, or 400,000 E80s per lane. There is a need for a design catalog for traffic volumes of 800,000 E80s and less, divided into appropriate smaller categories. The development of such a catalog, which uses granular base and subbase materials and bituminous surfacings, is discussed in this paper.

The catalog was developed with a mechanistic design procedure that was verified by comparison of the analytically determined performance with the actual field performance of a number of low-volume roads. The approach to the selection of materials for low-volume roads differs from other approaches since little relaxation of the material classification used for high-volume roads specified in the TRH 14 (2) is permitted. The adjustment for low-volume roads has been made in the design catalog.

First, the specification of materials used in the catalog is addressed. The mechanistic design procedure and the method used for its verification are also discussed, leading to a discussion on the compilation of the catalog. In conclusion, the catalog is compared with other low-volume road catalogs, such as Road Note 31 (*3*).

Materials

The TRH 14 (2) classification system for granular materials (G1 to G10) is used, which is summarized in Table 1. G1 material results from the crushing of fresh, unweathered rock. G2 material is composed of crushed, unweathered rock, but material other than the parent rock may be present. G3 material is a slightly lower-quality crushed stone. G4 to G6 materials are referred to as natural gravel; the quality of the gravel declines gradually from G4 to G6. G7 to G10 are referred to as gravel-soil or subgrade materials. Pavement structures for low-volume roads generally use G4 or G5 material as base and G5 or G6 material as subbase constructed directly on subgrades of various strengths. Crushed stone (G3/G2) materials are used as bases in low-volume road pavement structures when the design traffic volume exceeds 400,000 E80s.

Design Procedure

Development

The principal failure mode of granular pavements is permanent deformation (rutting). The pavement structures proposed in the catalog were therefore designed so that the pavement was considered failed once a rut of 20-mm depth developed on the surface of the pavement.

The *S-N* design method for granular materials was used to mechanistically analyze the pavement structures proposed for the catalog. The method was developed by Wolff (*4*) and is based on the use of *S-N* curves

TABLE 1 Granular Materials as Specified in TRH 14 (2)

CODE	MATERIAL	ABBREVIATED SPECIFICATIONS
G1	Graded crushed stone	Dense-graded unweathered crushed stone; max. size 37,5 mm*; compacted to 86-88 % of bulk density; PI < 4
G2	Graded crushed stone	Dense-graded unweathered crushed stone; CBR** > 80 @ 98 % mod. AASHTO density; max. size 37,5 mm; compacted to 100-102 % mod. AASHTO density; PI < 6
G3	Graded crushed stone	Dense-graded stone and soil binder; CBR > 80 @ 98 % mod. AASHTO density; max. size 37,5 mm; compacted to 98 % mod. AASHTO density; PI < 6
G4	Natural gravel	CBR > 80 @ 98 % mod. AASHTO density; PI < 6; max. size 53 mm
G5	Natural gravel	CBR > 45 @ 95 % mod. AASHTO density; PI < 10; max. size 63 mm or 2/3 layer thickness; grading modulus > 1,5
G6	Natural gravel	CBR > 25 @ 93 % mod. AASHTO density; PI < 12; max. size 63 mm or 2/3 layer thickness; grading modulus > 1,2
G7	Gravel-soil	CBR > 15 @ 93 % mod. AASHTO density; PI < 12; max. size 2/3 layer thickness; grading modulus > 0,75
G8	Gravel-soil	CBR > 10 @ in-situ density
G9	Gravel-soil	CBR > 7 @ in-situ density
G10	Gravel-soil	CBR > 3 @ in-situ density

* *1 inch = 25,4 mm*

** *Soaked CBR*

where S denotes a certain stress level or stress state and N denotes the number of stress repetitions to a failure condition. The method provides a design procedure and design curves for G1 to G6 materials. G7 to G10 are considered subgrade materials, and a design procedure based on vertical compressive strain in the subgrade, such as the South African mechanistic design method (5) was used to ensure adequate subgrade performance.

Rutting or permanent deformation may be caused by the elastoplastic behavior of granular materials (4). Elastoplastic behavior implies nonlinearity and inelasticity of the material, which means that

> the stress-strain curve of a granular material will not be a straight line with a constant slope, but a curved line with a slope or resilient modulus M_r varying with the applied stress, and plastic strain takes place during each application of a stress much smaller than the yield stress of the material and that these strains accumulate with load or stress repetitions. On the removal of stress, the loading stress-strain curve will not be retraced, but a hysteresis loop will form indicating the permanent deformation that took place during the application of each cycle of stress (4).

Therefore, it is important to use computer analyses that take account of nonlinear material behavior. The permanent deformation developed during each load cycle must be considered in the development of a design procedure for granular materials. The principle of S-N curves was found to be ideally suited for this purpose. S-N curves were developed from accurate measurements of elastic and plastic strains in granular layers under heavy vehicle simulator (HVS) testing of pavement structures, which incorporated such layers.

To develop a transfer function for rutting using the S-N curve principle, failure must be defined as a specific terminal permanent strain in the pavement layer for which the transfer function is being developed. However, any terminal permanent strain for the specific layer can be considered. Measuring permanent strain development with load repetitions in each of the separate layers constituting the pavement was made possible through the use of the multidepth deflectometer (6) in HVS testing. Data regarding the permanent strain in each layer caused by load (or stress) repetitions of a number of different wheel loads on various pavement types are available from HVS testing. These data are best presented by the following function of permanent strain versus repetitions of a specific wheel load (7):

$$y = (mx + a)(1 - e^{-bx}) \tag{1}$$

where

y = permanent strain,
x = number of load repetitions,
a, b, m = constants, and
e = base of natural logarithm.

The function is shown in Figure 1, which also shows the function $y = ax^b$ commonly used for modeling permanent deformation versus load repetition data in repeated-load triaxial testing. The lack of fit of the function $y = ax^b$ when applied to a large number of load repetitions is clearly demonstrated in Figure 1. Equation 1 can be used to calculate the number of repetitions of a specific wheel load (inducing a certain amount of stress) necessary to obtain the permanent strain in a specific layer that corresponds to the chosen terminal permanent strain. The stress induced in the layer by the wheel load can be calculated using the material parameters backcalculated from multidepth deflectometer measurements of elastic deflections taken during HVS testing. One point on a curve resembling an S-N curve can then be obtained by plotting an invariant of the stress (such as the sum of the principal stresses, θ) on the vertical axis and the number of load repetitions to failure on the horizontal axis. Additional points on the curve are obtained by repeating the exercise for other wheel loads.

Repeating the procedure for other HVS tests with different pavement structures and wheel loads but the same material allows a distribution of points to be obtained. The function that fits the data points best is referred to as the S-N curve or transfer function for that material at a 50 percent confidence level. The procedure is shown schematically in Figure 2.

Figure 3 shows S-N graphs for a G5 material at a 50 percent confidence level. Similar graphs were developed for G1, G2, G4, and G6 materials. For the G5 material in Figure 3, graphs are provided for a number of failure conditions, which are defined as a certain permanent strain of the layer (e.g., 10,000 $\mu\epsilon$). The stress state in the granular layer is denoted on the vertical axis by the

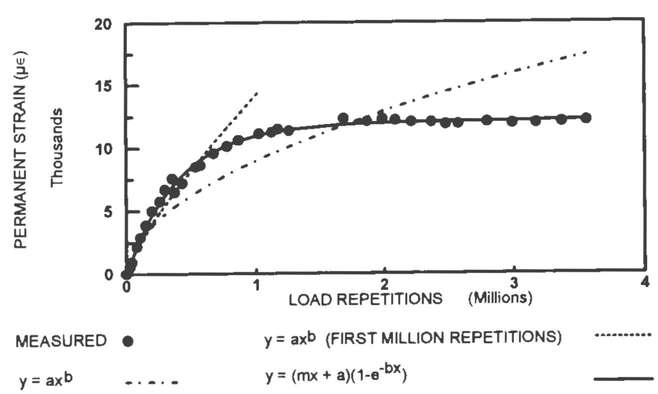


FIGURE 1 Different functions for modeling permanent strain development with load repetitions in granular materials.

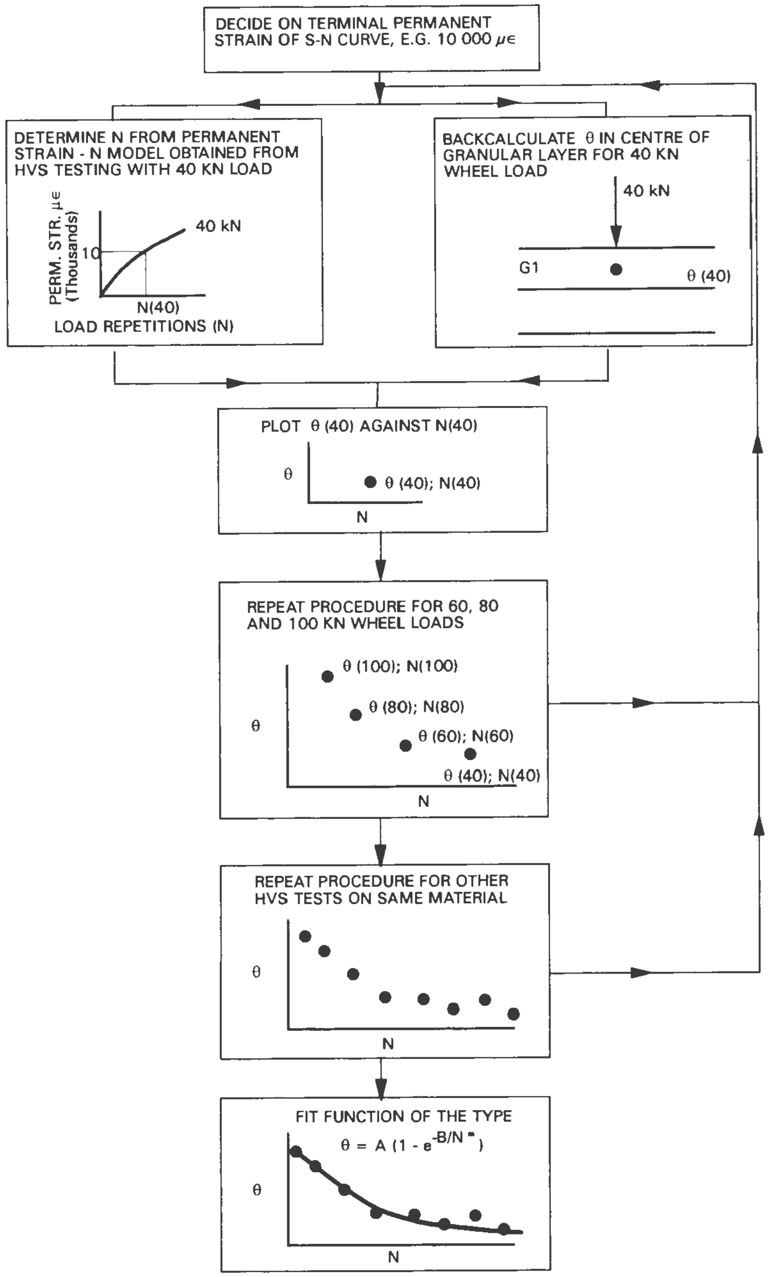


FIGURE 2 **Procedure for development of *S-N* curves from HVS data.**

sum of the principal stresses θ as calculated in the center of the layer. Figure 3, for example, indicates that 1.0×10^5 repetitions of a load that causes a stress state θ of 250 kPa (1 psi = 6.9 kPa) in a 150-mm-thick granular layer will cause a permanent strain of 10,000 $\mu\epsilon$ in the layer. A permanent strain of 10,000 $\mu\epsilon$ in a 150-mm-thick layer is equivalent to a permanent deformation or rut of 1.5 mm. This illustrates how *S-N* curves can be used to calculate the rut in a granular pavement layer when it is subjected to repetition of a standard wheel load. The surface rut in a granular pavement structure is then calculated by adding the permanent strains in each layer.

Verification

The *S-N* design method was verified by comparing the performance of low-volume road pavements with granular bases and subbases as determined from measure-

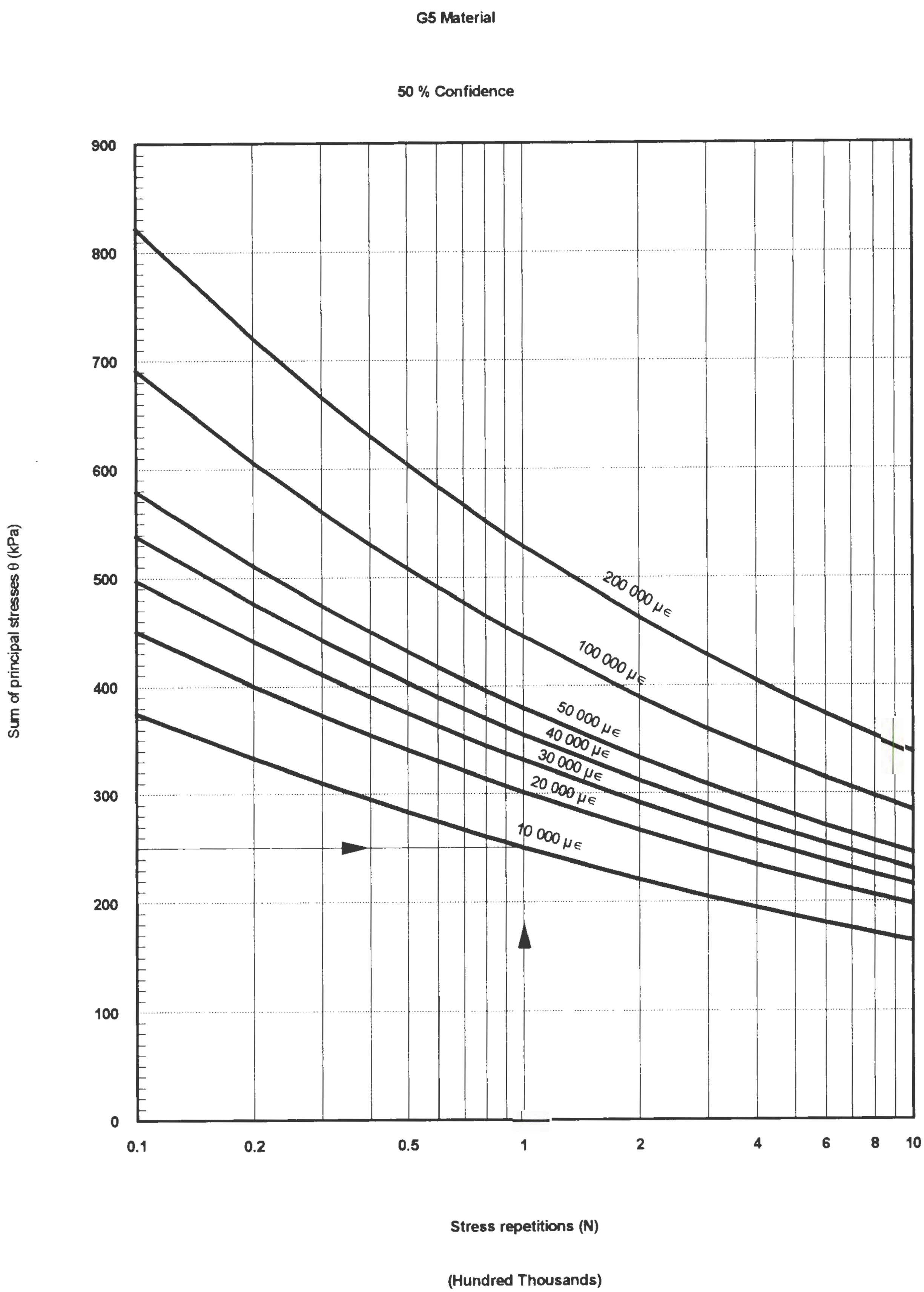


FIGURE 3 *S-N* graph for G5 material at 50 percent confidence level.

ments with performance predicted with the design method. Paige-Green compiled a data base (*8–16*) on the performance of 23 low-volume roads located in the provinces of the Orange Free State and the Transvaal in South Africa. The pavement compositions of the low-volume roads were determined from test pits, in situ density measurements, laboratory tests, and dynamic cone penetrometer (DCP) soundings. The tests were conducted in the outer and inner wheel paths as well as center of the road. The data were used to classify the pavement materials according to TRH 14 (*2*) standards. The pavement structures were analyzed with the *S-N* method to determine the number of standard wheel load repetitions (E80s) that would cause a 20-mm rut.

Paige-Green also measured ruts in the outer and inner wheel paths at the test positions and estimated the traffic volumes. The traffic was estimated from traffic counts conducted by the road authority. Results of these traffic counts are normally presented as average daily traffic (ADT) and percentage of heavy vehicles.

Studies on traffic composition on rural roads by the Transvaal Provincial Administration (TPA) indicated that a factor of 0.97 should be used to convert the number of heavy vehicles to E80s. The cumulative number of E80s over the pavement to the date of testing was determined from the following equation:

$$\text{E80s} = (\text{ADT}/2 \times \%\ \text{heavy vehicles} \times 0.97) * 365 * \text{age (years)}$$

The ADT was divided by 2 to get the traffic per lane (or per direction).

The data are summarized in Table 2. The number of E80s necessary to form a 20-mm rut was calculated from Paige-Green's data by linear extrapolation of the measured rut and traffic data. These data are also given in Table 2 and provide a conservative estimate of the carrying capacity of the pavement because the rate of permanent strain (rut) development with load applications (traffic) is not a constant (linear). Instead, it diminishes until it eventually approaches a constant value. The number of load repetitions where the rate of permanent strain approaches a constant value depends on the stress associated with the applied load, material type, and initial compaction of the pavement layer (*4*).

The number of E80s required to produce a 20-mm rut in the low-volume roads that were investigated was thus estimated from field measurements and also determined mechanistically with the *S-N* method and two other design methods that are currently used in South Africa. A comparison of the carrying capacities determined from the field measurements with those determined from the other design methods is shown in Figure 4. The *S-N* method predicts carrying capacities that are relatively close to the carrying capacities estimated from field measurements. The DCP method (*17*) predicts higher carrying capacities than those estimated from the field measurements. The factor of safety (FOS) method (*18,19*) gives the least consistent prediction of carrying capacity. It also has the poorest correlation with the carrying capacities estimated from field measurements compared with the other two design methods, perhaps because the FOS method is based on linear elastic theory.

A further comparison between carrying capacities determined from field measurements and mechanistically with the *S-N* method is shown in Figure 5. The *S-N* method predicts carrying capacities that are more conservative than the carrying capacities estimated from field measurements for the different pavement structures. The *S-N* method for the development of the catalog provides the best estimates of carrying capacity, and the carrying capacities calculated are slightly conservative.

Compilation of Catalog

The *S-N* method was used to compile the proposed design catalog for pavements with granular bases and subbases and bituminous surfacings for low-volume roads in moderate to dry and wet regions (Table 3). Traffic loading refers to the standard South African design axle, an 80-kN axle composed of two 40-kN double wheel assemblies (20 kN per tire at 520 kPa tire pressure). The designs are based on a failure criterion of 20 mm permanent deformation.

Elastic deflections on the surface of the proposed structures are important when deciding on the type of surfacing seal to be used. The design of the surfacing seals should receive careful attention since elastic deflections may be higher than normal (*20*). If maintenance is expected to be low, a 25-mm asphalt wearing course is recommended. The flexible low-volume road pavement structures cause relatively high horizontal tensile strains at the bottom of the thin asphalt wearing course, which leads to short fatigue lives for the asphalt. The asphalt fatigue lives were calculated and found to be adequate for the low traffic volumes.

A layer thickness of 150 mm was used for G4 to G6 materials because the material is usually obtained from borrow pits, and the difference in cost between a 100- and 150-mm layer is not excessive. If the material chosen has a more significant cost, such as crushed stone does, the use of a thinner layer was considered. However, layers less than 125 mm were considered impractical from a construction viewpoint.

The nonlinear elastic material characteristics for granular materials (G2 to G6) used in compilation of

TABLE 2 Summary of Data from Field Measurements on Low-Volume Roads (*8–16*)

ROAD NO.	PAVEMENT STRUCTURE	AGE (years)	N*	BASE PARENT MATERIAL	CUMM. TRAFFIC (E80's)	RUT (mm)	ESTIMATED TRAFFIC TO 20 mm RUT (E80's)
D514	125 G4**/200 G6/G8	6	2	Granite	$9{,}1\times10^4$	11	$1{,}7\times10^5$
D736	200 G6/100 G6/G7	6	2	Shale	$2{,}2\times10^4$	4	$1{,}1\times10^5$
D466-E	200 G5/200 G6/150 G6/G6	7	2	Laterite	$1{,}2\times10^5$	8	$1{,}8\times10^5$
D466-W	185 G5/100 G6/G8	7	2	Laterite	$1{,}6\times10^5$	6	$3{,}0\times10^5$
D390	175 G6/100 G6/G8	8	2	Laterite	$5{,}9\times10^4$	16	$5{,}2\times10^5$
D2485	200 G6/200 G6/G5	8	4	Shale	$1{,}5\times10^4$	6	$7{,}4\times10^4$
D804-8	200 G6/200 G6/G7	4	5	Andesite	$1{,}5\times10^4$	11	$4{,}9\times10^4$
D804-11	200 G5/200 G5/G5	4	5	Andesite	$1{,}5\times10^4$	12	$2{,}7\times10^4$
D804-19	200 G6/200 G6/G6	4	5	Calcrete	$1{,}2\times10^4$	2	$2{,}5\times10^4$
S191	150 G4/150 G5/150 G6/G6	9	3	Dolerite	$6{,}2\times10^4$	14	$1{,}2\times10^5$
S65-6	175 G4/175 G5/G7	11	3	Dolerite	$6{,}1\times10^4$	8	$8{,}9\times10^4$
S65-57	120 G4/200 G6/G6	11	3	Dolerite	$4{,}7\times10^4$	6	$1{,}5\times10^5$
S63	150 G4/200 G5/G5	10	3	Dolerite	$2{,}7\times10^4$	7	$1{,}6\times10^5$
P13/2	125 G4/150 G5/100 G6/G7	30	3	Dolerite	$1{,}9\times10^5$	12	$7{,}8\times10^4$
D467	150 G5/200 G6/G6	7	2,8	Norite	$6{,}4\times10^4$	8	$3{,}0\times10^5$
D540	200 G6/200 G6/G6	5	2,8	Laterite	$2{,}2\times10^4$	10	$1{,}6\times10^5$
D410-5	200 G5/200 G5/G5	3	5	Chert wad	$7{,}4\times10^3$	8	$4{,}4\times10^4$
D410-1	200 G5/200 G6/G5	7	5	Chert wad	$3{,}9\times10^4$	5	$1{,}9\times10^4$
P172-2	150 G4/200 G5/G6	7	5	Shale	$6{,}1\times10^4$	6	$1{,}6\times10^5$
D132	200 G5/150 G5/G7	4	5	Shale	$1{,}5\times10^4$	7	$2{,}0\times10^5$
D804	200 G4/200 G4/G5	7	5	Andesite	$3{,}5\times10^4$	5	$4{,}2\times10^4$
D404	140 G4/150 G6/G8	8	5	Shale	$9{,}8\times10^3$	6	$1{,}4\times10^5$
D421	125 G4/150 G5/200 G6/G7	4	1,8	Shale	$2{,}2\times10^4$	12	$3{,}2\times10^4$

* *Weinert climate factor N where N = (12* Evaporation in January/Mean Annual Precipitation)* (Weinert, H.H. *Basic Igneous Rocks in Road Foundations*. Research Report 218. National Institute for Road Research, CSIR, Pretoria, South Africa, 1964.)

** *Notation - 125 mm layer of G4 quality material. Bottom layer indicates in-situ material.*

the catalog were obtained from backcalculation of elastic deflections measured with the multidepth deflectometer (*4*). The resilient moduli for subgrade materials (G7 to G10) were taken from the South African mechanistic design method (*5*). The subgrade materials were considered to be linear elastic. The pavement structures for the moderate to dry regions were determined by using the resilient modulus values proposed by the South African mechanistic design method (*5*) for dry material. The pavement structures for the wet regions were determined by using the resilient modulus values proposed by the South African mechanistic design

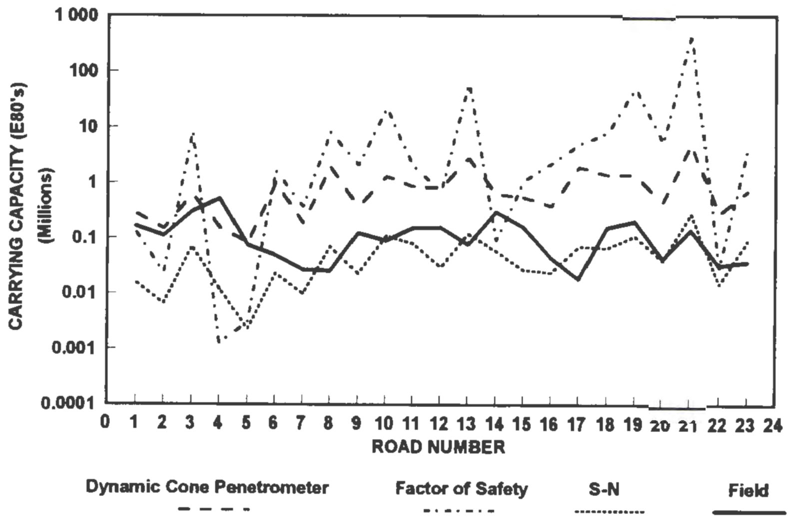


FIGURE 4 Comparison between carrying capacities of pavement structures determined from field measurements and with various design methods.

method (*5*) for wet material. The material properties used in the calculations are given in Table 4.

The pavements are assumed to be supported by in situ material with a CBR of at least 3 (G10). Layers shown in the catalog with lower strength than the in situ subgrade may be omitted if there is adequate strength for the total pavement depth.

The materials in the catalog are classified by their soaked bearing strength in terms of CBR. However, the proposed pavement structures relate to performance at field moisture content. This is the same approach used in the TRH 4 (*1*) catalog. When existing roads are upgraded, the pavement materials of the existing road must be classified in terms of the soaked CBR to relate to the catalog. Procedures to accomplish this with the DCP and other test methods have been documented (*21*).

Comparison With Other Catalogs

The TRH 4 (*1*) design catalog was used as the basis for comparison. The structures proposed in the TRH 4 cat-

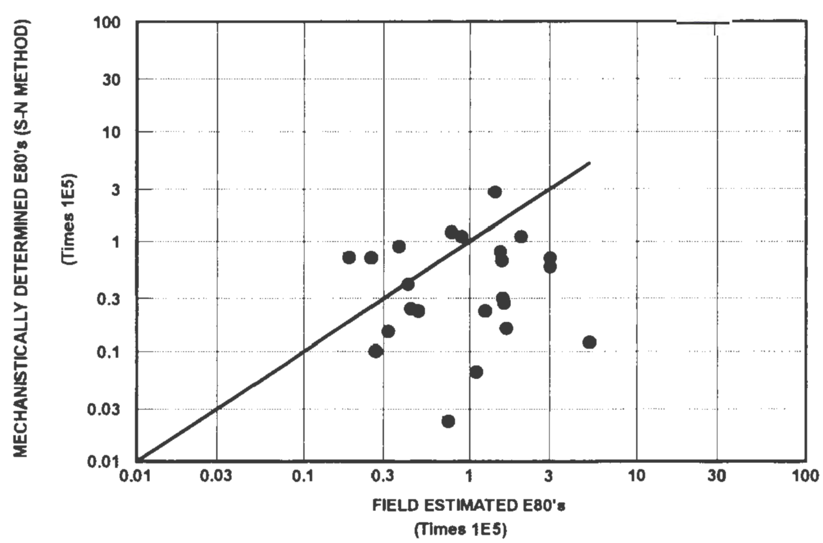


FIGURE 5 Comparison between field-estimated and mechanistically calculated carrying capacities of various low-volume roads.

TABLE 3 Proposed Design Catalog for Low-Volume Roads with Granular Bases and Subbases Compiled with *S-N* Design Method

TRAFFIC CLASS	TRAFFIC (80's)	PAVEMENT STRUCTURES		
		MODERATE TO DRY REGIONS	WET REGIONS	LOW MAINTENANCE
E0-1	< 5 000	# 150 G6* 150 G8 150 G9 G10**	150 G5 150 G7 150 G9 G10	25 A+ 150 G6 G10
E0-2	5 000 - 30 000	150 G5 150 G7 150 G9 G10	150 G4 150 G6 150 G8 G10	25 A 150 G6 150 G7 G10
E0-3	30 000 - 100 000	150 G4 150 G6 150 G8 G10	150 G4 150 G5 150 G6 150 G7 G10	25 A 150 G5 150 G9 G10
E0-4	100 000 - 200 000	150 G4 150 G5 150 G8 G10	150 G3 150 G6 150 G9 G10	25 A 150 G4 150 G9 G10
E1-1	200 000 - 400 000	150 G4 150 G5 150 G7 150 G9 G10	150 G3 150 G6 150 G8 G10	25 A 150 G4 150 G8 G10
E1-2	400 000 - 800 000	125 G2 150 G6 150 G9 G10	125 G2 150 G5 150 G9 G10	25 A 150 G4 150 G5 150 G8 G10

\# *Double surface treatments assumed on all pavement structures unless otherwise indicated.*

* *Notation - 150 mm layer of G6 quality material.*

** ***Pavement assumed to be supported by in-situ material having a CBR of not less than 3 (G10) and semi-infinite depth. Layers shown in the catalogue with lower strength than the in-situ subgrade may therefore be omitted provided that adequate strength exists for the total pavement depth.***

\+ *25 mm asphalt.*

alog for design traffic volumes of 2×10^5 and 8×10^5 E80s in a moderate to dry region were compared with structures from TPA catalogs (E. G. Kleyn, unpublished data) and Road Note 31 (*3*).

Comparisons between design catalogs are not always straightforward. Traffic classes and material classification frequently do not correspond; interpolation and approximation are required. The comparison described in this paper is no exception. The TPA and Road Note 31 material specifications had to be approximated to TRH 14 standards. The traffic classes do not correspond directly either. However, an effort was made to include most of the relevant data to give a sense of how the different catalogs compare.

The comparison is shown in Table 5. Comparison of pavement structures for design traffic volumes of 200,000 to 300,000 E80s indicates that the 100-mm G4 material proposed as base by the TRH 4 catalog may not be adequate. The TPA and Road Note 31 designs require approximately the same quality and thickness base and subbase material as the *S-N* method. However, for the same performance, the *S-N* method requires fewer selected layers and the use of lower-quality material than the other catalogs do.

Comparing pavement structures proposed by the catalogs for design traffic volumes of 800,000 to 1 million E80s indicates that the 100-mm G4 material proposed as base by the TRH 4 catalog may not be adequate.

TABLE 4 Material Properties Used for Calculations with MICHPAVE in Mechanistic Analysis of Granular Pavement Structure Proposed for Design Catalog (in Imperial units)

MATERIAL TYPE	K_0*	K_1** (psi)@	K_2#	POISSON RATIO	COHESION (psi)	ϕ+ (degrees)	DENSITY (pcf) (kg/m³)
G2	0,80	17 994	0,35	0,20	0	45	140 2242
G4	0,75	2717	0,44	0,25	0	45	135 2162
G5	0,70	9611	0,22	0,33	0	43	130 2082
G6	0,60	6848	0,36	0,35	0	40	130 2082
MATERIAL TYPE	K_0	MODULUS OF ELASTICITY (psi)		POISSON RATIO			DENSITY (pcf) (kg/m³)
		DRY	WET				
G7	0,50	34 783	17 391	0,35	-	-	110 1762
G8	0,50	26 087	13 043	0,35	-	-	110 1762
G9	0,50	20 290	10 145	0,35	-	-	110 1762
G10	0,50	13 043	6 522	0,35	-	-	110 1762

* *Earth pressure coefficient used in MICHPAVE.*

** *Value of K_1 in the equation $M_r = K_1\, \Theta^{K2}$ used in MICHPAVE to describe the non-linearity of granular materials.*

@ *Program written for imperial units (1 psi = 6,9 kPa).*

\# *Value of K_2 in the equation $M_r = K_1\, \Theta^{K2}$ used in MICHPAVE to describe the non-linearity of granular materials.*

\+ *Angle of internal friction from the Mohr-Coulomb failure theory.*

The base layer required by the TPA and Road Note 31 designs is thicker but of a slightly poorer quality material than that of the *S-N* method design. The TPA and *S-N* method designs require similar subbase layers, which are of a poorer-quality material than the subbase layers required by the TRH 4 and Road Note 31 designs. The subbase required by the Road Note 31 design is also thicker than the subbases required by the other catalogs. For the same performance, the *S-N* method requires fewer selected layers constructed of a lower-quality material than the other catalogs.

The catalog compares well with most current design catalogs, except for the TRH 4 catalog designs, which may be inadequate in the E0 and E1 design traffic classes.

Conclusions and Recommendations

The development of a design catalog for low-volume roads using granular materials and bituminous surfacings is described. The materials used in the catalog are described in the standard TRH 14 (2) material specification that is widely used in South Africa. *S-N* design method used for the development of the catalog is briefly discussed. Important aspects concerning the design method include the following:

- The method was developed with data accumulated from HVS testing of in situ pavements incorporating granular layers.
- The method is based on elastoplastic material behavior, which implies nonlinearity and the accumulation of plastic strain with each load cycle.
- A function that accurately fits permanent deformation compared with load repetition data for tests with a large number of load repetitions [$y = (mx + a)(1 - e^{-bx})$] was used in the development of the model.

The design method was verified by comparing the expected life of 23 low-volume road pavement structures

TABLE 5 Comparison Between *S-N* Method and TRH 4, TPA, and Road Note 31 Catalogs

CATALOGUE	DESIGN TRAFFIC (E80's)	STRUCTURE
TRH 4 (CSRA, 1985a)	200 000	100 G4* 125 G5 150 G7 150 G9 #
	800 000	100 G4 150 G5 150 G7 150 G9
TPA (Kleyn, unpublished data)	300 000	150 G3/G4 150 G6 150 G8 150 G9
	1 000 000	150 G3 150 G6 150 G7 150 G8
ROAD NOTE 31** (TRL, 1992)	330 000	150 G3/G4 150 G5/G6 200 G7
	769 000	150 G3/G4 200 G5/G6 200 G7
S-N METHOD (Wolff, 1992)	200 000	150 G4 150 G5 150 G8
	800 000	125 G2 150 G6 150 G9

* *Notation - 100 mm layer of G4 quality material.*

\# *Subgrade CBR of more than 3 assumed.*

** *Catalogue actually specifies GB1 to GB3 as base material. GB3 was selected as base material.*

Note: *Material codes approximated to TRH 14 standards.*

as determined from field measurements with the expected life of the same pavement structures as determined mechanistically with the *S-N* design method. The pavement lives calculated were found to compare relatively well.

The catalog was compiled on the following basis:

- The designs are based on the standard 80-kN axle load;
- A 20-mm rut was used as failure criterion;
- Layer thickness of 150 mm were generally used, unless the cost of the layer was significant;
- The pavements are supported on a subgrade with a CBR in excess of 3; and
- The pavement structures relate to performance at field moisture content.

Comparison with other low-volume road catalogs indicated that for the same performance, the *S-N* method designs require construction of fewer selected layers and lower-quality material than designs proposed by the other catalogs. Allowance was made for differences in material specification and loading equivalency in the comparison of the different catalogs.

The use of the catalog is recommended in the design and upgrading of roads in the South African environment.

ACKNOWLEDGMENTS

This paper is based on research done by the Division of Roads and Transport Technology (DRTT) of the CSIR for the Department of Transport. The Director General of the Department of Transport and the Director of DRTT are thanked for their permission to publish this paper.

REFERENCES

1. *Structural Design of Interurban and Rural Road Pavements.* TRH 4. Technical Recommendations for High-

ways, Committee of State Road Authorities, Pretoria, South Africa, 1985.

2. *Guidelines for Road Construction Materials.* TRH 14. Technical Recommendations for Highways, Committee of State Road Authorities, Pretoria, South Africa, 1985.
3. *A guide to the Structural Design of Bitumen-Surfaced Roads in Tropical and Sub-Tropical Countries.* Road Note 31 (Draft 4th Edition). Transport Research Laboratory, Department of Transport, London, United Kingdom, 1992.
4. Wolff, H. *The Elasto-Plastic Behaviour of Granular Pavement Layers in South Africa.* Ph.D. dissertation. Department of Civil Engineering, University of Pretoria, Pretoria, South Africa, 1992.
5. *The Mechanistic Design Method Used to Evaluate the Pavement Structures in the Catalogue of the Draft TRH 4 1980.* Technical Report RP/2/81. National Institute for Transport and Road Research, CSIR, Pretoria, South Africa, 1981.
6. De Beer, M., E. Horak, and A. T. Visser. The Multi-Depth Deflectometer (MDD) System for Determining the Effective Elastic Moduli of Pavement Layers. In *First International Symposium on Non-Destructive Testing of Pavements and Back Calculation of Moduli,* ASTM STP 1026, ASTM, Philadelphia, Pa., 1988.
7. Wolff, H., A. T. Visser, E. G. Kleyn, and M. De Beer. The Elasto-Plastic Behaviour of Crushed Stone Bases in Flexible Pavement Structures. In *Proc., Seventh International Conference on Asphalt Pavements, Nottingham, United Kingdom,* 1992.
8. Paige-Green, P. *The Use of Marginal Base Course Materials in Some Low Volume Roads in the Orange Free State.* Interim Report IR 88/033/1. South African Roads Board, Research and Development Advisory Committee, Pretoria, South Africa, 1991.
9. Paige-Green, P. *The Use of Marginal Base Course Materials in Three Low Volume Roads in the Transvaal.* Interim Report IR 88/033/2. South African Roads Board, Research and Development Advisory Committee, Pretoria, South Africa, 1991.
10. Paige-Green, P. *The Use of Marginal Base Course Materials in Three Low Volume Roads in the Western Transvaal.* Interim Report IR 88/033/3. South African Roads Board, Research and Development Advisory Committee, Pretoria, South Africa, 1991.
11. Paige-Green, P. *The Use of Marginal Base Course Materials in Some Low Volume Roads in the Transvaal.* Interim Report IR 88/033/4. South African Roads Board, Research and Development Advisory Committee, Pretoria, South Africa, 1991.
12. Paige-Green, P. *Interim Recommendations on the Use of Marginal Base Course Materials in Low Volume Roads.* Interim Report IR 88/033/5. South African Roads Board, Research and Development Advisory Committee, Pretoria, South Africa, 1991.
13. Paige-Green, P. *The Use of Marginal Base Course Materials in Three Low Volume Roads in the Eastern Transvaal.* Interim Report IR 91/201/1. South African Roads Board, Research and Development Advisory Committee, Pretoria, South Africa, 1991.
14. Paige-Green, P. *The Use of Marginal Base Course Materials in Three Sections of Low Volume Roads in the Eastern and Western Transvaal.* Interim Report IR 91/201/2. South African Roads Board, Research and Development Advisory Committee, Pretoria, South Africa, 1991.
15. Paige-Green, P. *The Use of Marginal Base Course Materials in Three Sections of Low Volume Roads in the Western Transvaal.* Interim Report IR 91/201/3. South African Roads Board, Research and Development Advisory Committee, Pretoria, South Africa, 1992.
16. Paige-Green, P. *Interim Recommendations on the Use of Marginal Base Course Materials in Low Volume Roads in the Transvaal.* Interim Report IR 91/201/4. South African Roads Board, Research and Development Advisory Committee, Pretoria, South Africa, 1992.
17. Kleyn, E. G. *Aspects of Pavement Evaluation and Design as Determined with the Dynamic Cone Penetrometer (DCP)* (in Afrikaans). M.Sc. thesis. Department of Civil Engineering, University of Pretoria, South Africa, 1984.
18. Maree, J. H. *Design Parameters for Crushed Stone in Pavements* (in Afrikaans). M.Sc. thesis. Department of Civil Engineering, University of Pretoria, Pretoria, South Africa, 1978.
19. Maree, J. H. *Aspects of the Design and Behaviour of Road Pavements with Granular Bases* (in Afrikaans). Ph.D. dissertation. Department of Civil Engineering, University of Pretoria, Pretoria, South Africa, 1982.
20. *Appropriate Standards for Bituminous Surfacings for Low Volume Roads.* Manual 10. Southern African Bitumen and Tar Association, Roggebaai, South Africa, 1992.
21. *Towards Appropriate Standards for Rural Roads: Discussion Document.* Department of Transport, Pretoria, South Africa, 1992.

Economic Design of Bridges on Low-Volume Roads in Southeast Alaska

James Scott Groenier, *USDA Forest Service, Petersburg, Alaska*

The USDA Forest Service in the Tongass National Forest has a unique task in designing bridges for low-volume roads in remote areas of southeast Alaska. This paper focuses on an overview of the methodology for the location, design, construction, and maintenance of these bridges with an emphasis on economics. The site of the bridge is located during planning of the low-volume road. Forest roads have an average daily traffic of less than 50 vehicles per day. A few of the considerations in determining the location of the bridge are preliminary hydrology and hydraulics, stream-bed strata, and environment. Other factors may also control the location and design of the bridge. The bridge is designed after the site has been located. The typical structure is designed for use by a standard U80 logging truck with an L90 yarder overload. The type and size of structure will be based on economics, design life, and environmental limitations of construction materials. The bridge will also be designed with ease of construction in mind. Construction methodology is a major consideration in design because of the remoteness of the area and limited construction equipment. Typical construction materials in southeast Alaska are steel, wood, and concrete. Concrete is not readily available and requires more maintenance with the heavy logging equipment that uses these structures. The last concern is maintenance after the bridge has been completed. The structures should have low maintenance or be easily repaired with limited equipment and resources.

The USDA Forest Service in the Tongass National Forest has a unique task in designing bridges for low-volume roads in remote areas of southeast Alaska. The remoteness of the island archipelago limits the type of structure that can be used and the method of construction for these bridges. The location, design, construction, and maintenance of these remote bridges are of great economic concern with ever-decreasing budgets.

Bridges are the single highest cost items to be built on any road construction project. The cost of one structure can often exceed the construction cost of a mile of road, which is approximately $160,000 per mile in southeast Alaska. Damage done to bridges and major drainage structures such as large pipes greater than 2.44 m (8 ft) from inadequate location, poor design, or poor installation can result in high maintenance costs or structural failures. The loss of a structure can result in major erosion problems, severe economic impacts, and even the loss of human life. The USDA Forest Service, Region 10, Tongass National Forest, Stikine Area, uses the following approach to bridge location, design, economic analysis, construction, and maintenance.

Bridge Location

During initial layout on aerial photographs, the site of the structure crossing is considered a critical control in the planning of the low-volume road. Several issues are concerns in determining the location of a bridge in the field: preliminary engineering, preliminary hydrology and hydraulics, horizontal and vertical alignment, and environmental concerns such as ice flow, stream-bed strata, fisheries, and wildlife.

Preliminary Engineering

The preliminary field location of the bridge should be determined by an experienced road locator with input from internal resource groups and other government agencies. Concerns from agencies outside the Forest Service, such as the Alaska Department of Fish and Game, the U.S. Army Corps of Engineers, and the U.S. Coast Guard, can control where the proposed structure may be located according to regulations and laws. Resource specialists' concerns within the Forest Service also have to be addressed in defining the site location. Intradepartmental resource groups include archaeologists, fishery biologists, wildlife biologists, hydrologists, ecologists, soil scientists, geotechnical engineers, and bridge engineers. Their comments and suggestions help in locating a site that is acceptable to all parties concerned.

Preliminary engineering in the form of site reconnaissance and site drainage analysis is one of the most important items in a bridge site investigation. Analysis should be completed before the site survey to ensure a good design. Topographic site surveys require accurate information about existing highwater marks, current edge of water, bottom of stream bank, top of stream bank, bedrock outcropping, and other important topographical items. The survey should have horizontal and vertical control to the nearest tenth of a foot, and it must include a benchmark and reference points for future construction needs. Stringent procedures will help ensure that the contour map developed, whether by hand or on a computer model, will also be accurate.

Preliminary Hydrology and Hydraulics

Major drainage crossings and other geographical control points are normally located before connecting the crossings with the best location possible. This ensures that the size and number of crossings are kept to a minimum. Preliminary hydrology is completed before field review or site survey if possible. If the contract survey is completed ahead of time, a topographic site survey is conducted on the proposed location, and a site map is constructed from the survey data and used to calculate hydraulics. The hydrology is modeled using three methods: the synthetic hydrograph (L. Bartos, USDA Forest Service), which is valid for drainage areas < 3.88 km^2 (1.5 mi^2); the Water Resource Atlas by Ott Water Engineers, which is valid for drainage areas with a minimum of 2.59 km^2 (1 mi^2); and the R10 FLOWMOD, which is a hydrologic model for estimating ungaged stream flows on the Tongass and Chugach national forests, to determine the drainage area runoff for a 50-year design event. The channel hydraulics are modeled using Mannings equations or the Xspro channel cross-section analyzer by Grant et al.

The flow volumes and correlating elevations from the hydrology and hydraulic programs are then compared, and an educated estimate is made for the 50-year flow and the 50-year highwater elevation. This information and the contour map are taken into the field for a site reconnaissance of the proposed bridge. The site reconnaissance is very important in validating the hydrological and hydraulic models. The bridge engineer needs this experience with the site when designing the bridge in the office. The engineer must consider crossing alignment, roadway alignment, bridge clearances, and channel stability.

Horizontal and Vertical Alignment

The most efficient alignment is achieved when the stream channel and roadway alignment crossing is perpendicular. This is not always possible. If a skewed bridge is required, the skew should be minimized as much as possible to reduce specialized structure costs.

Two major concerns regarding structure location are horizontal and vertical alignment, which depend greatly upon the design speed of the road. Horizontal geometry of the road must be compatible with a straight bridge because of off-tracking of the logging truck's rear wheels. The structure location must provide a short tangent section on both approaches to the bridge to prevent the rear wheels from off-tracking to the inside of the turn and contacting the bridge superstructure. Vertical alignment deals mainly with stopping sight distance (SSD) for sag and crest curves. To reduce the required length of bridge, many bridges designed in southeast Alaska are located in sag curves. Another critical concern with sag curves is the g-force that a fully loaded truck exerts on a structure in the apex of a sag curve. SSD is also the major concern for crest curves as narrow, single-lane bridges require additional sight distance to allow vehicles to be seen.

Environmental and Geological Concerns

Waterway clearances are important to the bridge superstructure to prevent debris or ice damage during high flows. Those at the site should check for signs of damage to nearby trees from floating debris and ice flows. Highwater marks from spring and fall flooding should also be monitored. It is important to consider elevation, snow accumulation, and the location of the drainage basin in relation to northerly or southerly facing slopes relative to spring and fall floods when waterway opening needs are determined. The area of the watershed that will be affected by roads and commercial activities, as well as the associated increase in runoff and debris associated with this development, must also be considered.

If the channel is currently stable, not agrading or degrading, it is important to design a structure that will leave the streambed stable. Scour potential for the structure site and variation with the streambed strata should be considered. The potential for scour typically decreases as the size of the substrate in the stream increases; sand and gravel have the greatest potential for scour, and bedrock has the least. Streambed strata can also help verify the estimated velocity of the stream during flooding. The average size of the material in the stream will help approximate the velocity of the water flowing at peak flows using a chart that plots the relationship of water velocity and stone weight.

Design

After the stream crossing has been determined, design procedure is based on structure type, materials, site requirements and economics, and environmental limitations. Ease of construction and equipment needs are significant during the design procedure.

Structure Type and Size

The type and size of bridge depend on the type of traffic that will use the structure. On the Tongass National Forest, structures are designed for single-lane and double-lane classifications of roadway, and they are often not accessible to public traffic. A timber sale bridge is typically a single-lane 4.88 m wide (16-ft) bridge designed for a U80 logging truck of 72 574.8 kg (80 tons) gross vehicle weight (GVW) (see Figure 1) with an L90 yarder overload of 81 646.6 kg (90 tons) GVW (see Figure 1). The general public bridges are designed for the same loading conditions, but may be one or two lanes wide [4.88 m (16 ft) to 7.32 m (24 ft) wide]. Structure design life is dependent on structure use (i.e., short-term installation of 5 to 10 years or long-term installation of 50 years). If modular structures are used as mobile bridges, site use may be considered as short term for less than 5 years, but actual structure design life will be long term, approximately 35 to 50 years.

Material

Timber, steel, and concrete are used for construction (see Figure 2 for typical bridge cross sections). Numerous design considerations, such as economics, longevity, strength, and span lengths, are used to determine which material type should be used at a specific location.

Timber is broken down into two categories: treated timber and native log stringers. Native log stringers are used for short-term bridges preferably constructed of sitka spruce or yellow cedar that is found on or near the bridge site. If neither species is available, western hemlock can be used, although this will lead to a shorter design life. The spans for these bridges typically range from 9.14 m (30 ft) to 18.29 m (60 ft), with some spans reaching 30.48 m (100 ft). Only high-quality logs may be used to minimize failure because of natural defects. Treated timber bridges, which consist of glulam beams and panelized decks, are long-term structures with spans of up to 36.58 m (120 ft). Currently, wood is cost-effective, but as the price of wood increases, the cost of steel and concrete should be reviewed.

Steel bridges are usually considered long-term bridges when they are painted with a USDA Forest Service Region 10 System 6 paint system. These bridges can be permanent or temporary structures. Permanent steel bridges are plate girder bridges with economical spans from 24.38 m (80 ft) to 36.58 m (120 ft). Bridges that cannot be easily moved after initial installation are considered permanent structures. Modular steel bridges that can be moved with less effort are considered long-term structures and used as temporary or permanent structures. These bridges range in size from 9.14 m (30 ft) to 24.38 m (80 ft); some bridges of up to 33.53 m (110 ft) are being built. Modular bridges are limited by the size and weight of the bridges, as well as the size of equipment available for installation. Modular bridges are usually constructed as half sections, with the two sections bolted together in-place. In contrast, permanent bridges come in many pieces and have to be totally assembled on site. Modular steel bridges are more economical than plate girder bridges because less labor is required on site for construction.

Concrete is seldom used in Southeast Alaska due to cost and handling problems. Concrete bridges can be precast, prestressed beams, or cast-in-place slabs. Precast beams are more economical than cast-in-place slabs, since the price of concrete at a remote site costs as much as $765 per meters cubed ($1000/yd^3) in-place. Precast prestressed beams are more economical, but it

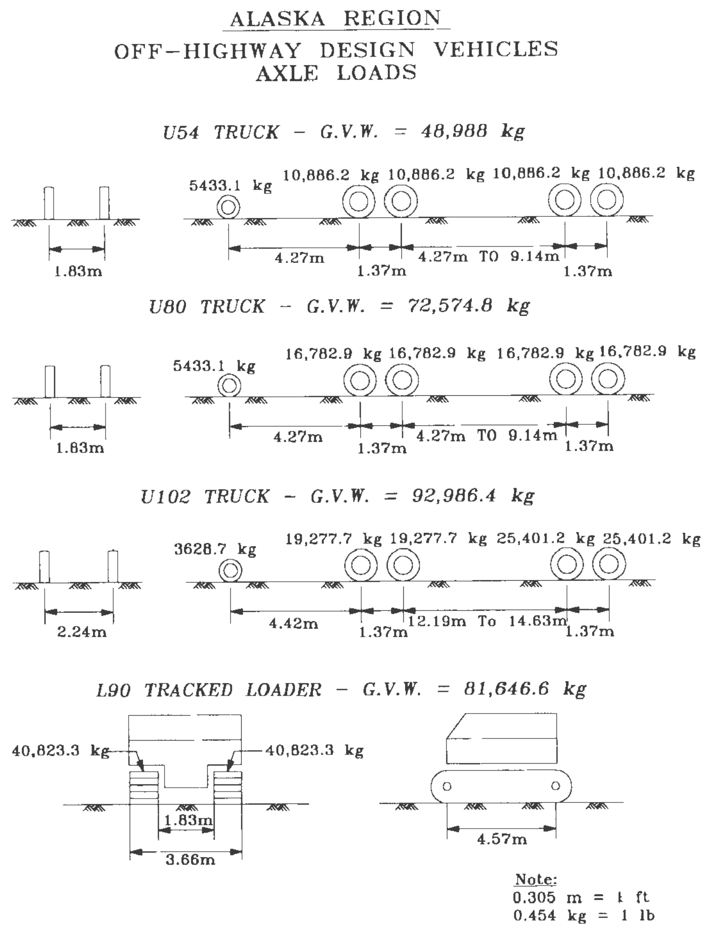


FIGURE 1 Axle loads for off-highway design vehicles in the Alaska region.

is difficult to transport them along low-volume forest roads without damaging them.

Site Requirements

Several factors must be considered in the design of a structure, including debris clearance, alignment, erosion and scour protection, substructure design needs, and span requirements. Debris such as ice flow during spring breakup and floating debris such as submerged logs and stumps that have a potential to cause damage and erosion must be cleared. Horizontal and vertical alignment is required. Scour protection should be designed to reduce or eliminate possible substructure failure, which could eventually result in the total failure of the bridge. This can be accomplished by using a chart that plots the 50-year flood water velocity against the riprap weight and uses this average size as a minimum.

The most important site requirement is substructure design. Without an adequate substructure, the bridge has a high probability of failure. Basic substructures are mud sills, log cribs, gravity wirewalls (i.e., welded wirewalls and geogrid walls), and pile bents with timber lagging for fill containment. Substructure design is site-specific. Span requirements can be single-, double-, and multispan structures. The bridges built in southeast Alaska are typically single spans because of floating debris problems and small drainage system requirements. Midstream pier construction is usually avoided because of erosion problems and associated risks to the superstructure.

Economic Analysis

An economic analysis compares the costs of materials, freight, and installation for superstructure and substructure for equivalent length bridges. All bridges will have a clear span of approximately 15.24 m (50 ft) and be built on mud sills at a site on Kuiu Island in the Tongass National Forest. Costs will be presented for four bridge

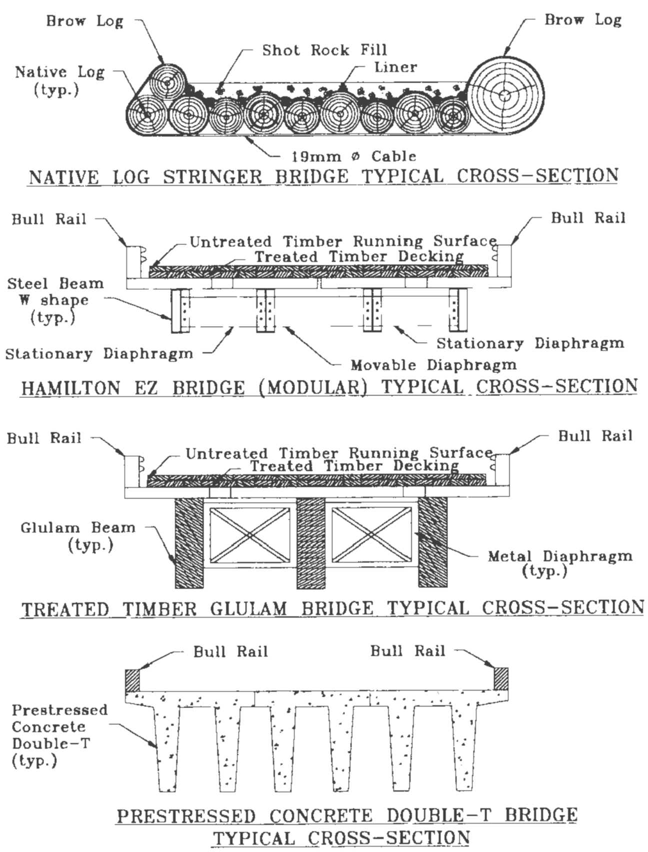


FIGURE 2 Typical bridge cross sections.

types: 15.24-m (50-ft) native log stringer, 14.33-m (47-ft) treated timber glulam, 15.24-m (50-ft) hamilton EZ (modular), and 15.24-m (50-ft) precast prestressed concrete double-T bridge. Plate girder structures are not economical at this site. These costs do not include profit and risk costs and are estimated to be constructed under timber sale contract with no Davis-Bacon wage regulations. Bridges of this length can be built with standard logging equipment; no specialized equipment is required. The excavation process is approximately the same for all four structures and has been neglected in the cost calculations.

Native Log Stringer Bridge

The costs of a native log stringer bridge do not include freight if the log stringers are found at the site. The material costs have a realized value or "stumpage value" of $85/m^3 ($200/1000 bf) using Scribner Decimal C Scales to the Forest Service. The current market value for sitka spruce in dimensional lumber is $1060/m^3 ($2500/100 bf). Superstructure installation includes placing 10 to .91 m (36 in.) diameter × 15.24-m (50-ft) long stringers with brow logs, wrapping and tightening cables, placing geotextile fabric, and spreading surface rock. Substructure installation includes notching sill longs and setting native log sills. The following calculations apply:

Item	*Cost ($)*	
Superstructure	Stumpage	Market
Materials	8,240	105,250
Freight	n/a	n/a
Installation	2,640	2,460
Superstructure total	10,880	107,710
Substructure		
Materials	250	3,130
Freight	n/a	n/a
Installation	860	860
Substructure total	1,110	3,990
Total cost	11,990	111,600

Treated Timber Glulam Bridge

Freight costs include transport by barge from Seattle, Washington, to southeast Alaska and mobilization to the bridge site. Substructure freight for mobilization to the site is included with superstructure freight costs. Superstructure installation includes setting glulam beams, placing and fastening panelized decking, bullrails, and running planks. Substructure installation includes placing sills, constructing backwalls, and backfilling behind backwalls. The following calculations apply:

Item	*Cost ($)*
Superstructure	
Materials	32,000
Freight	4,940
Installation	7,070
Superstructure total	44,010
Substructure	
Materials	12,000
Freight	730
Installation	4,000
Substructure total	16,730
Total cost	60,740

Hamilton EZ Bridge

Freight costs include transportation from Springfield, Oregon, to Seattle, Washington; transport by barge from Seattle to southeast Alaska; and mobilization to the bridge site. Substructure freight for mobilization to the site is included with superstructure freight costs. Superstructure installation includes setting half sections and bolting them together. Substructure installation includes placing sills, constructing backwalls, and backfilling behind backwalls. The following calculations apply:

Item	*Cost ($)*
Superstructure	
Materials	33,600
Freight	8,400
Installation	3,260
Superstructure total	45,260
Substructure	
Materials	4,200
Freight	320
Installation	4,000
Substructure total	8,520
Total cost	53,780

Precast Prestressed Concrete Double-T

Freight costs include transport by barge from Seattle to southeast Alaska and mobilization to the bridge site. Superstructure installation includes setting concrete beams, grouting keyways, and placing and fastening bullrails. Substructure installation includes placing sills, placing backwalls, and backfilling behind backwalls. Concrete structures are more brittle than timber or steel structures. Therefore, they require twice the time for mobilization. The following calculations apply:

Item	*Cost ($)*
Superstructure	
Materials	33,650
Freight	5,370
Installation	2,740
Superstructure total	41,760
Substructure	
Materials	9,690
Freight	3,170
Installation	1,670
Substructure total	14,530
Total cost	56,290

Cost Comparison for 50-Year Life Cycle

A present-worth cost analysis will be calculated for these four structures for a 50-year life cycle with no assumed salvage value at the end of the cycle. The present-worth method converts future dollars to present dollars. A 4 percent discount rate will be used for the analysis for long-term investments, in accordance with Forest Service policy.

The Hamilton EZ and treated timber bridges will be constructed at year zero, and the running surfaces will be replaced at years 10, 20, 30, and 40 at a cost of $5,000 per redecking. The concrete bridge will also be constructed at year zero and will be redecked at year 25 at a cost of $32,000. The native log stringer bridge will be constructed four times, at years 0, 12, 25, and 37. The following calculations apply:

1. Native Log Stringer (stumpage value)
Cost = −11,990 − 11,990(p/f, 4%, 12) − 11,990(p/f, 4%, 25) − 1,990(f/p, 4%, 37) = −$26,785

2. Hamilton EZ
Cost = −53,780 − 5,000(p/f, 4%, 10) − 5,000(p/f, 4%, 20) − 5,000(p/f, 4%, 30) − 5,000(p/f, 4%, 40) = −$62,023

3. Precast prestressed concrete double-T
Cost = −56,290 − 32,000(p/f, 4%, 25) = −$68,293

4. Treated timber glulam
Cost = −60,740 − 5,000(p/f, 4%, 10) − 5,000(p/f, 4%, 20) − 5,000(p/f, 4%, 30) − 5,000(p/f, 4%, 40) = −$68,983

5. Native log stringer (market value)
Cost = −111,600 − 111,600(p/f, 4%, 12) − 111,600(p/f, 4%, 25) − 111,600(p/f, 4%, 37) = −$249,314

The cost analysis shows the native log stringer bridge to be the best alternative. However, $85/m^3 ($200/1000 bf) is not a realistic value for the price of wood. The value of wood should be greater than the stumpage value, but possibly not as high as the market value. Another problem with native log stringers is availability of stringer. In time, the required high-grade timber for these structures may not be available. The most economical structure is the Hamilton EZ bridge, with a total present-worth cost of $62,023. The present-worth costs for other structures were approximately $6,000 greater than that of the Hamilton EZ bridge. This cost analysis was conducted for a 15.24-m (50-ft) structure, which represents the typical size bridge built in southeast Alaska for stream crossings. Results may differ for longer or shorter span structures.

Construction

The method of construction is a major consideration in the design of the bridge because of the remoteness of the area and the limited construction equipment. The best structures for construction are made of materials that are somewhat forgiving, such as steel and wood. Plans must accurately reflect the site conditions so that the structure is constructed at the designated location as designed. Occasionally, bridges are installed without plans—these eventually fail. Frequently, a change in site conditions will require reassessment of the original design to ensure that the proposed structure is still viable. The last and most important item is to have qualified inspectors on site during construction activities to monitor work and progress. Without adequate inspection, the structure could appear sound but have internal problems that could surface as maintenance problems, shortened structure life, or ultimately structure failure.

Maintenance and Inspection Program

Maintenance after the bridge is complete is the final concern. Structures should be designed and constructed to require low maintenance. Even with a low maintenance structure, a thorough inspection and maintenance program must be developed to ensure the integrity of all bridges. The program should be based on structure type and have an inspection and maintenance frequency not to exceed two years. This strenuous approach will extend the life of bridges. If maintenance is required, the structure will be easily repairable with limited equipment.

Inspection involves reviewing all structure components including the load rating of each structure. The frequency is mandated by National Bridge Inspection Standards (NBIS) to not exceed 2 years. With Native Log Stringers and fracture critical bridges, the frequency may need to be increased to yearly.

The inspections should be conducted by trained personnel familiar with local structure types. Load rating of structures should be completed for all native log stringer bridges more than 5 years old and for all other structures more than 10 years old. Some structures may need to be load rated earlier. Posting structures to ensure safety of contract operators and the general public should be a priority if load ratings indicate such a need.

Conclusion

The procedure for engineering bridges for southeast Alaska is a unique problem that must take many factors into consideration, including location, design, construction, and maintenance. These are important in determining the end product, which is the most economic bridge for low-volume roads in southeast Alaska that meets expected design life requirements.

CONSTRUCTION

Lightweight Fill Materials for Road Construction

Guy Kohlnhofer, *Braun Intertec Corporation*

A lightweight fill is any material used to replace a heavier in situ soil to reduce the load on subgrade soils. The use of lightweight fills is increasing in many areas of the United States. Various materials have been used; however, because of their experimental nature there are no formal design guidelines specific to each material. Additional research is required to determine more specific design guidelines for each available lightweight fill material. The selection of a lightweight fill is generally based on local knowledge of each fill material. For any construction project requiring a lightweight fill, there may be several alternative materials to consider. However, due to a limited knowledge or unfamiliarity with these materials, design engineers might not consider them as an alternative. This synthesis has been written to help determine what technology and materials are available for use in road construction in areas where a lightweight fill material may be appropriate. The fills investigated are sawdust and wood chips, expanded polystyrene, foamed concrete, and shredded tires.

Minnesota is known as the Land of 10,000 Lakes; however, there are just as many, if not more, swamps and bogs. Generally, the soils near the swamps and bogs restrict the type of road construction available. These construction restrictions can cause city, county, and state engineers a variety of design problems. Sometimes traversing these areas can be costly and time consuming. Consequently, new roads may not follow the most desirable routes, and existing routes may continue to have high maintenance costs.

The use of wetlands for construction purposes is regulated, making it difficult to obtain construction permits. These regulations complicate the use of wetlands beyond the scope of this report.

In Minnesota, soils of concern are generally organic or clay materials and involve a relatively high water table near the surface. When encountering areas of organic materials or other poor soils, the first alternative is to find an acceptable route around the area. In some instances, this may not be feasible and the poor soils must be traversed. When constructing on an area of poor soils, there are several alternatives:

1. The soils of concern can be removed to a depth where stronger soils are found and backfilled with a granular material;
2. A small amount of the poor soil may be removed and replaced with a lightweight material of lower density, thus creating a floating platform;
3. If the in situ soils are only a concern while saturated, the area may be permanently drained to a suitable depth;
4. Construction may be staged and the soils may be surcharged;
5. A geotextile may be placed over the existing vegetation and covered with a granular material; and
6. The area may be bridged by a suspended structure.

These options are all viable and should be evaluated individually to determine feasibility and cost-effectiveness. This report will focus on the second alternative—construction with lightweight materials.

Lightweight Fills

Various lightweight fill materials have been in use for many years throughout Minnesota. The materials most commonly used are sawdust and wood chips and chunks. These materials are light and inexpensive by-products and are often readily available. Although many of their qualities are desirable for a fill material in road construction, wood products also possess some undesirable qualities. In many cases, wood products placed above the permanent water tables are highly biodegradable. Additionally, toxins may leach from the wood products and enter the groundwater system. A danger of fire with wood products also exists, more so during than after construction.

Shredded waste tires are another readily available and inexpensive product that may be used as a lightweight fill material. The use of tires below the water level causes concern for toxic leachates when exposed to certain chemicals. This is currently a concern of the Minnesota Pollution Control Agency (MPCA), and further independent research is welcome.

Expanded polystyrene (EPS) is a form of heavy-duty Styrofoam that has a density lower than any other lightweight fill currently in use. EPS is nonbiodegradable and is easily placed using little or no machinery. The deterrent of EPS is its high cost. Although widely used and relatively inexpensive in Norway, there is limited documented use in the United States. One documented use is in Pickford, Michigan, near a Dow Chemical plant. The costs of EPS in Pickford are unique in that the Dow plant had a stockpile of unused EPS at the time it was needed.

Bark and Sawdust

Bark and sawdust are lightweight fill materials that will greatly reduce the static load of a pavement structure. Wood products have been used for many years by the timber companies as a lightweight fill material for the reclamation of peat bogs and swamplands. Many county, state, and federal agencies have also used these products as lightweight fill material for swamps, as well as in areas where excessive soil loads could cause slope shear failures.

Cosmopolis, Washington (1)

The use of sawdust was researched by the FHWA in 1974 on a site near Cosmopolis, Washington, a site at which excessive rain fall led to a slope failure (*1*). Although these soils were *above the water table*, sawdust was selected as a lightweight fill. This created a problem in that when placed above the water table (an aerobic condition), sawdust is biodegradable and will decay and break down. To reduce the biodegradation process, an emulsified asphalt sealer was mixed with the sawdust on the slopes, creating an impermeable shell to help achieve a near-anaerobic condition. Three varieties of sawdust were used: hog fuel, planer chips, and bark chips. The uncompacted and compacted densities were 380 kg/m^3 (23.7 lb/ft^3) and 581 kg/m^3 (36.3 lb/ft^3), respectively. Traffic characteristics of this roadway are not reported; therefore, the effects of heavy loads cannot be determined. The FHWA report states the following conclusions:

1. Sawdust is a very workable and easily placed fill material;
2. Sawdust can reduce the driving weight of a potential slide area by as much as 71 percent, thereby reducing the chance of slope shear failure;
3. The fibrous intertwining of sawdust particles tends to create an interconnected web, thereby distributing the loads in a more lateral direction;
4. Sawdust does not require mechanical compaction —the compaction obtained from the construction equipment is adequate;
5. Indications are that sawdust fill materials can sustain roadway sections for 15 years or longer before the decomposition of the fill requires reconstruction; and
6. The use of sawdust material above the water table should be based on economics, availability, and environmental concerns.

Mt. Baker–Snoqualmie National Forest, Washington (2)

Sawdust and wood waste have been used within the Mt. Baker–Snoqualmie National Forest for varying purposes from fill over soft, compressible soils to backfill behind and on top of fabric-reinforced retaining structures. Multiple road structures have been built since 1975 using some type of wood waste product during construction. Many of these sites were located along steep slopes experiencing slope instability and slump. The existing soils were subcut to depths of up to 4.3 m (14 ft) and replaced with sawdust. The sawdust was placed in 0.3-m (1 ft) layers and compacted with hauling and spreading equipment. The surface of the sawdust was sometimes sprayed with emulsified asphalt to prevent ultraviolet degradation; however, many sawdust piles 20 years or more old showed little degradation, and therefore not all sawdust construction sites received asphalt protection.

The sites constructed within the Mt. Baker–Snoqualmie National Forest are performing well with little additional settlement. The method of using sawdust in areas of marginal stability has proven cost-effective and easy to construct. Additional construction sites have been planned and will be monitored.

The Minnesota Department of Transportation (DOT) does not have written design guidelines for the use of wood products in road construction. The Minnesota DOT Office of Materials and Research sponsored a research study in 1976 conducted by Lukanen (*3*). This project consisted of six methods of floating roadway widening sections over a peat swamp:

1. Corduroy—a method of placing tied logs perpendicular to the road to create a solid working platform for further construction;
2. Wood chip working platform—a modification of the corduroy method to create a working platform with 0.6 m (2 ft) of wood chips; the chips are then capped with a minimum of 15 cm (6 in.) of clay to reduce their exposure to air;
3. Wood chip embankment—used in areas of high fill to reduce the total fill weight and capped with 0.6 m (2 ft) of clay to reduce exposure to air;
4. Full-width fabric section—fabric is placed transversely across the road and into the ditches; it should limit horizontal movement and transfer some of the load of the widening to the existing roadbed;
5. Ditch fabric section—fabric is placed 5.2 m (17 ft) from centerline and extended into the ditch area, adding tensile strength and allowing a thinner layer of fill on which equipment can work; and
6. Weakened plane section—a 1.1-m-deep (3 ft), 15-cm-wide (6 in.) trench is dug 4.6 m (15 ft) from centerline, allowing the widened section to settle without causing distress within the original road section.

The construction and observation of these six methods yielded the following findings and conclusions.

> Wood chips provide a good working platform upon which fill can be placed and light machinery can operate. They did not displace in front of machinery or fill and had sufficient stability to withstand submersion in water. Running water, however, may easily displace wood chips. . . . Any kind of disturbance of the existing vegetation mat appears to cause problems. A drainage ditch was cut into the root mat and longitudinal cracking appeared in the lane adjacent to the ditch. A drainage ditch was cut adjacent to the widening over a shallow swamp, and longitudinal cracking appeared in the pavement for several hundred feet along the ditch. Later borings showed the fill to be setting on .9 to 1.5 meters (3-5 ft) of peat in this area. . . . The construction costs of floated widenings are much less than the keying method of sobcutting the sides to a stable subgrade and backfilling. Floating widenings are also much quicker to construct than keyed widenings. . . . Any drainage ditches that have to be constructed in conjunction with widening on future projects should be placed a sufficient distance away from the road to prevent transverse movement and the formation of longitudinal cracks in the pavement. (*3*)

When using sawmill residue or timber corduroy as a road embankment, the biodeterioration of the lignocellulosic material has to be controlled. Wood is made up of more than 50 percent cellulose and 10 percent to 35 percent lignin. Lignin is a more complex carbohydrate than cellulose; however, both will break down under aerobic conditions. Various methods to control biodeterioration exist:

- Ensure that the wood product remains below the groundwater table;
- Use chemical treatments such as an emulsified asphalt or other type of sealer, which may be expensive and difficult to distribute evenly;
- Use a geotextile fabric to restrict aeration to the sawmill residue (*4*); and
- Cap wood product with a layer of plastic soil to reduce exposure to air.

Polystyrene Foam

Expanded polystyrene has had limited use in the United States as a lightweight fill. (One documented case was in Pickford, Michigan.) However, Norway and Finland, with their numerous wetlands and peat bogs, have extensive experience with EPS as a lightweight fill. For many years, Norway has been using EPS as insulation boards for a frost protectant. In 1972, Norway began using polystyrene in greater thicknesses to reduce excessive settlements in areas where soils were unable to support the static load of a pavement structure.

The cost of EPS depends on the strength required, amount needed, and various other factors. For this reason, it is impossible to estimate product cost accurately. The unit cost of EPS may be high although an analysis of the cost of load reduction per square foot must be conducted for all alternatives under consideration. Total load reduction is the weight of the soil removed minus the weight of the fill used. The cost of load reduction takes into account the cost of the soil excavation plus the cost of the required fill material. Although a material of lower density may be more expensive, it may require less excavation, thereby decreasing the cost of excavation and the amount of fill material required.

Pickford, Michigan (5)

Engineers within the United States have done little experimentation with polystyrene as a lightweight fill. One application of EPS is located near Pickford, Michigan, on the Big Munuscong River. This is a case where the soft dense soils, approximately 2002 kg/m^3 (125 lb/ft^3), of a bridge approach were sliding toward the river, moving the piers and abutments. Since increasing the length of the bridge or completely removing the unstable soils were determined to be too costly, various lightweight fills were considered.

Lightweight expanded aggregate would have required excavation below the pier and river bottom. This option was eliminated because of instability problems and the transportation cost of hauling the aggregate (nearest source was 402 km (250 mi) away).

A seminested series of corrugated metal pipes in a sand fill was considered, which would have decreased the fill load by creating void areas within the pipes. This procedure would have required 4.9 m (16 ft) of excavation and 305 lineal m (1,000 ft) of various culvert sizes. Because of the cost of the corrugated pipe, this option was not chosen.

Another option, cedar logs, would have provided a load reduction of 1121 kg/m^2 for each 1 m of fill (70 lb/ft^2 for each 1 ft). Since this option would require 3823 m^3 (5,000 yd^3) of cedar logs, it was rejected because the logs could not be obtained within a reasonable time. Additionally, their long-term durability was questionable.

Polystyrene was examined and found to weigh much less than sand or very soft clay, 48 kg/m^3 versus 1922 kg/m^3 (3 lb/ft^3 vs. 120 lb/ft^3). The tremendous weight reduction per depth of excavation would be a great advantage, reducing the required excavation substantially. EPS has long-term durability and is only slightly affected by the surrounding moisture (5). Approximately 983 m^3 (1,285 yd^3) of EPS would be required.

The Michigan Department of State Highways and Transportation (DSHT) discovered that due to an ordering error Dow Chemical Company (Midland, Mich.) possessed a large amount of EPS from a previous order and provided the material at a lower than normal unit cost. To obtain a product cost for future projects, a producer would have to be contacted and given the amount and required strength of the needed EPS.

The unit cost of the EPS for this project is not a representative cost for future projects. Polystyrene fill was selected because of its low density, minimum required excavation, durability, ease of placement, immediate availability, and comparatively low cost. The foam came in 2.44-m × 0.61-m × 3.81-cm boards (8 ft × 2 ft × 1.5 in.). These boards were taped together in bundles of eight forming a billet 2.44 m × 0.61 m × 0.31 m (8 ft × 2 ft × 1 ft) weighing 22.7 kg (50 lb). The billets were placed, staggering joints, at a total thickness of 1.52 m (5 ft) near the bridge where the most load reduction was needed to one billet of 3.81 cm (1.5 in.), 45.7 m (150 ft) from the abutment. It was important to cover the polystyrene with enough fill material to resist the buoyant forces of the foam when the Big Munuscong River rises in the spring. Prior to covering with soil, the foam was covered with a 4-mil-thick polystyrene sheeting. This sheeting was placed to protect the foam from possible spillage of petroleum-based liquids that could seep through the granular cover and dissolve the polystyrene.

Norway

Numerous projects have been conducted throughout Norway using expanded polystyrene as a lightweight fill. Norway, like the northern United States, experiences numerous freeze-thaw periods and extended periods with temperatures below freezing. The United States may be able to benefit from some of Norway's experience with EPS.

In an issue of *Veglaboratoriet*, produced by the Norwegian Road Research Laboratory, the following experiences with expanded polystyrene as a lightweight fill are discussed (6).

Compressive Strength

The compressive strength of the EPS was tested at four construction sites using excavated samples. These construction sites in Solbotmoan, Flom, Langhus, and Lenken have been in place from 1 to 5 years. The study concluded that the compressive strength of the expanded polystyrene has not decreased measurably over time. In fact, in some cases the compressive strength had increased, possibly due to the increase of the moisture content of the fill material.

Moisture Resistance

In the most extreme case, a polystyrene fill was permanently submerged below the groundwater level for 9 years at Solbotmoan to evaluate EPS resistance to moisture. An increase of 9 percent absorption was observed. Above the groundwater level, the moisture content decreased to 1 percent by volume. In areas of periodic submersal, a moisture content of 4 percent was observed. Due to these moisture contents a density of 16.6 kg/m^3 (28 lb/yd^3) is recommended for dry conditions and 33.2 kg/m^3 (56 lb/yd^3) for submerged conditions.

Settlements

Only slight settlements have been noticed at a few sites throughout Norway. Most settlements can be attributed to slippage of the underlying embankment materials. Settlement within the polystyrene blocks can be expected to be between 0.5 percent and 2 percent of the thickness of the blocks.

Flammability

Similar to sawdust and bark chips, polystyrene is flammable. Two sites in Norway, a fill and a stockpile, accidently burned and were destroyed. The fill was a 1499-m^3 (1,960-yd^3) polystyrene embankment leading up to Knatten Bridge in the district of Akefhsus. The fire was caused by welding too close to the bridge abutment. It is recommended that extreme care be taken when conducting any high-temperature work near polystyrene.

A self-extinguishing polystyrene is available at a higher cost; however, statistics show it would be cheaper to use the standard polystyrene and accept the occasional loss.

Chemical Resistance

Petroleum-based products will react with and dissolve polystyrene foam. Therefore, it is necessary to cover the material with a concrete slab or a petroleum-resistant geotextile rather than an asphalt surface. In the event of an accident or spill, the chemical products would be prevented from reaching the EPS.

Pavement Bearing Capacity

Through analysis conducted at the Norwegian Road Research Laboratory, the stresses and strains in the pavement and subgrade were calculated using a modified Chevron program. The research concluded that varying the depth of the polystyrene fill had no significant influence on the pavement design.

A 10-cm (4-in.) concrete slab is recommended below the bituminous surface. If the concrete slab is omitted, the bituminous would need to be increased by 0.3 to 0.4 m (12 in. to 16 in.) to keep stresses and deflections at the same level. This is a substantial structure, and the increase cost would need to be considered in a cost comparison study.

Dynamic Loads

If the total dynamic and static loads are limited to 80 percent or less of the compressive strength of the polystyrene, EPS can theoretically service an unlimited number of loads. Therefore, for all practical purposes of a 20-year design period, dynamic loads do not affect an expanded polystyrene fill. It is likely the pavement surface will reach the end of its design life long before the EPS fill material. EPS is available in a wide range of compressive strengths; these strengths must be known for each loading condition.

Icing Problems

EPS has high insulative properties. During a frost cycle, a pavement structure with EPS does not experience the warming effects of the soil below causing differential icing with respect to the surrounding road surface. This may cause traffic hazards; therefore deicing procedures must be established.

Shredded Tires

Automotive tires are discarded at a rate of 3 million [84 106 m^3 of tires (110,000 yd^3)] annually in just the state of Minnesota (7). This large volume of waste has little applicable reuse. In 1985, a recycling firm and a logging contractor contacted the MPCA regarding the use of waste tires in constructing forest roads. In 1986, the Hedbom Forest Road in Floodwood, Minnesota, was constructed using waste tires. The tires were placed below the base material using nine different placement strategies (7). These placements ranged from whole tires tied together to spreading shredded tires as a base material. As of the last review in 1989, all of the test sections were performing exceptionally well. This study caused the MPCA to conduct testing to establish some guidelines for the use of waste tires as a fill material. These guidelines have led to some controversy. Many individuals wish to further investigate the use of waste tires as a lightweight fill as opposed to stockpiling, which may have equal or greater hazards.

MPCA Guidelines

The use of shredded tires as a lightweight fill may be efficient and effective, but there is an environmental concern. A study published by the Minnesota Pollution Control Agency discusses many of the possible negative aspects of the use of shredded tires as a fill material (*8*). Laboratory tests were conducted over a range of pH levels from 3.5 to 8 to simulate potential worst-case scenarios. These laboratory tests were extreme and may or may not represent actual field conditions. Soils in northeastern Minnesota tend to be acidic, while the soils in the southwest are primarily alkaline. Soil sampling was also conducted around two existing waste tire stockpiles. The laboratory tests conducted and soil samples tested yielded the following conclusions:

1. Toxic metals are leached, in the highest concentrations, from the tires under acidic conditions. The ma-

terials of concern are barium, cadmium, chromium, lead, selenium, and zinc.

2. Polynuclear aromatic hydrocarbons (PAHs) and total petroleum hydrocarbons are leached from tire materials in the highest concentration under alkaline conditions.

3. Asphalt materials may leach higher concentrations of contaminants of concern than tire materials under some conditions.

4. Recommended allowable limits for drinking water set by the Minnesota Department of Health may be exceeded under "worst-case" conditions for arsenic, barium, cadmium, chromium, lead, selenium, and carcinogenic and noncarcinogenic PAHs. "Worst-case" conditions for metals appear to occur at low pH (acid) conditions. "Worst-case" conditions for organics appear to occur at high pH (basic) conditions.

5. A biological field survey did not identify significant differences between waste tire areas and control areas with respect to soil samples.

6. Potential environmental impacts from the use of waste tires can be minimized by placing tire materials only in the unsaturated zone (above the water table) of the roadway subgrade. This can be accomplished by placing alternative materials, such as wood chips or soil, below the water table.

7. The metals leached from waste tire stockpiles are similar in concentrations to those leached in areas where tires are used for fill.

From this study, the MPCA has set forth guidelines for using shredded tires as a lightweight fill material in construction. Although other states may vary, following are the Minnesota guidelines (9).

1. Road Repair and Construction:

- Shredded waste tires can be used in road construction or repair if the tire shreds will be above the water table and not in contact with groundwater. Tire shreds cannot be used below the water table.
- Roads and road slopes must be designed and constructed to reduce water infiltration and to promote surface water drainage away from the roadbed, to minimize the amount of surface water seeping through the shredded tires.

2. General Construction (applies to *all* shredded tire construction projects)

- A synthetic geotextile fabric is recommended above and below the areas where shredded waste tires are used. The fabric will prevent movement of soil into the tire shreds and will hold the tire shreds in place.
- Tire shreds must be covered by a low-permeability surface to reduce seepage of surface water.

If a proposed construction project meets these criteria, the MPCA should be contacted to determine what type of information or monitoring is required. If subsequent monitoring indicates unacceptable types and levels of leachates, it is likely the MPCA will require the removal of the fill material.

CSAH 21, Rice, Minnesota (10)

A portion of County State Aid Highway (CSAH) 21 in Benton County, north of Rice, Minnesota, crosses a swamp. A traffic count conducted in 1987 indicated the annual average daily traffic was approximately 500 vehicles. Increases in the level of the swamp caused the need to increase the roadway elevation. Conventional methods of additional granular fill were used to raise this roadway above the water level. During extended periods of dry conditions, the underlying peat material decreased in strength and failed. A decision was made to use a lightweight fill rather than remove the soft underlying soils. In an experiment to reduce a waste problem while solving a construction problem, shredded tires obtained from a tire landfill, which the MPCA had designated for early cleanup, were selected as a lightweight fill material.

The existing embankment was excavated to a level 15 cm (6 in.) above the surrounding marsh. A layer of shredded tires, approximately 1.2 m (4 ft) in depth, was sandwiched between a geotextile fabric and covered with 1.1 m (3.5 ft) of clean granular backfill. The geofabric was used to prevent the intrusion of soils into the tire fill. The section was paved with gravel base, subbase, and bituminous as normal and is performing well with no apparent visual problems.

Duluth and Tower Avenues, Prior Lake, Minnesota (11)

The intersection of Duluth Avenue and Tower Avenue in the city of Prior Lake required reconstruction in 1991. To obtain state funding, the city was required to change the alignment of the two roadways. The new alignment crossed a wetland with organic deposits approximately 9.1 m (30 ft) in depth.
Three options were considered:

1. Excavate to an adequate load bearing strata and refill with granular material,
2. Stage construction and surcharge to compress the underlying organics, and
3. Replace subgrade with a lightweight fill.

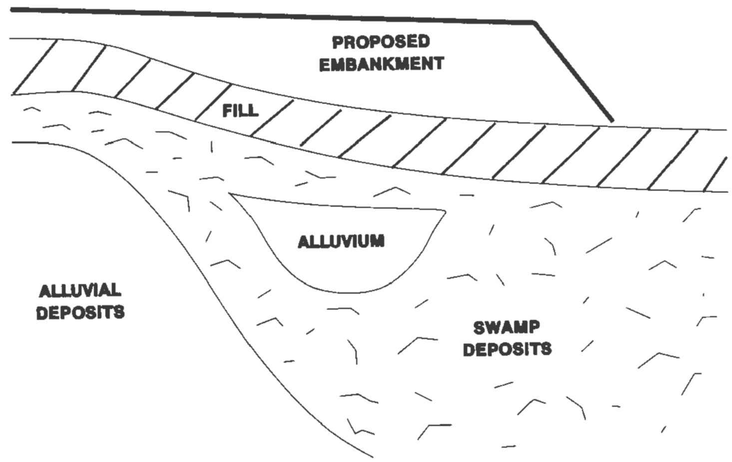


FIGURE 1 Soil cross section.

Because of the relatively higher costs and time constraints of Options 1 and 2, the city decided on Option 3, construction with a lightweight fill. Wood chips were placed to a depth of 0.3 m (1 ft) above the anticipated groundwater level on top of a Minnesota DOT Type 3 geotextile fabric. Wood chips were placed to conform with the MPCA regulation restricting the use of shredded tires below the groundwater level. To reduce the risk of biodegradation, wood chips were not used for the entire fill. Approximately 3 ft of shredded recycled tires were placed directly on the wood chips and covered with a Minnesota DOT Type 5 geotextile. A 15-cm (6-in.) sand layer and 61 cm (24 in.) of Class 5 were used to cover the entire section prior to paving. A cross section of the existing soils and the final design may be found in Figures 1 and 2.

During placement of the 0.76 m (2.5 ft) of sand and Class 5 materials, the tire shreds settled 15 cm (6 in.), resulting in the need for an additional 15 cm of granular base. Although this settlement was unexpected, the additional 15 cm of compaction increased the modulus of the tire shreds; however, the actual increase in modulus is unknown. Based on this project, it is the city engineers' opinion that pavement design using shredded tire subgrades should assume an R-value of 5 for the tire material (*11*).

This section was constructed in 1991, so it is too early to determine if any problems or irregularities will develop. The section appears to be in good condition and the city engineers expected it to perform adequately.

Interim Design Guidelines

This interim report was conducted to determine some of the characteristics of a shredded tire fill and was generated from data gathered from a private access road constructed with shredded tires at thicknesses of 0.9 m and 1.8 m (3 ft and 6 ft). Some of the conclusions found in this interim report are as follows:

1. The rate and effectiveness of compaction are similar to sawdust fills.
2. It appears all appreciable increase in bulk unit weight can be achieved with about 24 passes of a D7 Caterpillar.
3. The maximum bulk unit weight of tire shreds with an average particle area of 929 cm^2 (1 ft^2) is approximately 320 kg/m^3 to 368 kg/m^3 (20 lb/ft^3 to 23 lb/ft^3).

Currently, the primary cost of using waste tires is for the transportation and placement of the material. Many stockpilers are willing to shred and give the tires away to reduce their inventory. As the use of waste tires increases, the cost from the suppliers may increase; however, it will likely remain one of the least expensive materials.

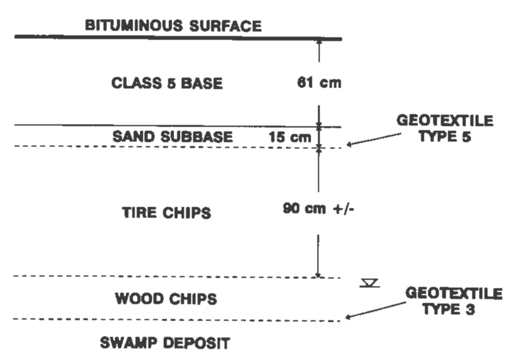


FIGURE 2 Final design.

Foamed Concrete

Lightweight foamed concrete has been widely used in building construction for floors, roof decks, and insulation. Recently, foamed concrete has been used as a lightweight fill in highway construction, primarily in the area of bridge abutments. These areas may contain poor soils and require a lightweight fill that does not exhibit horizontal pressures on the adjoining structure. Foamed concrete has been used for many roadway projects simply because a lightweight fill material with high strength is required.

Foamed concrete is generally mixed on site by the supplier with a portable mixing unit. The cost of this fill may be too costly—at \$30 to \$35/m^3 (\$40 to \$45/yd^3)—for small county and city projects in remote areas; although for special cases, such as bridge abutments, it may be an economical alternative.

The only foamed concrete considered to be a "lightweight" fill for this report is a product called Elastizell, produced and patented by the Elastizell Corporation of America, based in Ann Arbor, Michigan. The material mix plants are truck mounted and are therefore available in Minnesota. Various classes of the material are produced with different strengths and densities. The maximum cast densities can be found in Table 1.

7th and Lyndale, Minneapolis, Minnesota

A large project in Minneapolis, Minnesota—the intersection of Lyndale Avenue and 7th Street near I-94—was constructed using Elastizell. (A detailed report of this project is not available.) A new bridge over I-94 required high approach fills. Normally this would not have been a problem; however, the approach was located over poor soils and was to cross over the Basset Creek storm sewer tunnel. Future settlement of the high fill was a concern. A fill was needed that was light enough to minimize settlement and strong enough to support the high approach with minimal horizontal stress. Elastizell was selected and various classes with

TABLE 1 Elastizell Specifications

CLASS	MAXIMUM CAST DENSITY (kg/m^3)	MINIMUM COMPRESSIVE STRENGTH (pa) x 1,000	ULTIMATE BEARING CAPACITY (kg/m^3) x 1,000
I	384	68.9	6.8
II	481	276	28.3
III	577	552	56.6
IV	673	827	83.9
V	801	1,103	112
VI	1,281	2,068	210

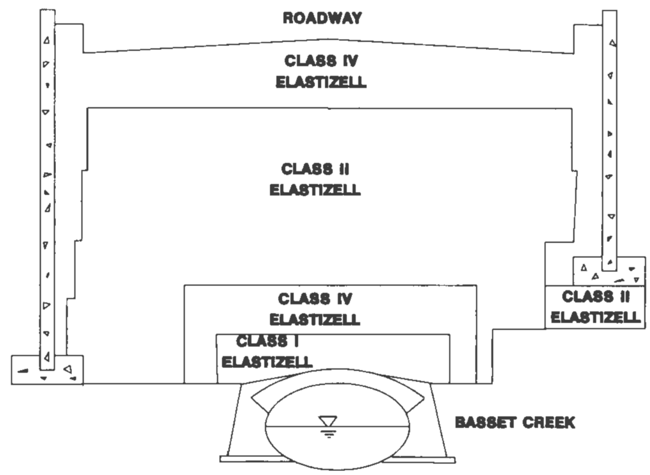


FIGURE 3 Elastizell cross section.

different compressive strengths were used. A cross section of the various materials is shown in Figure 3. This project has performed well for 10 years and no future problems are anticipated.

Waiska River, MI-28 Crossing (12)

During the replacement of the bridge superstructure crossing the Waiska River 9.6 km (6 mi) west of I-75 in Michigan (1976), it was discovered that the eastern approach had settled 0.46 m (1.5 ft) and would require replacement. Because of the soft alluvial clay subsoils, either the bridge structure would have to be lengthened or the soils in the "approach area" would have to be removed and replaced with a lightweight fill material. The relatively high cost of lengthening the bridge structure was prohibitive and a lightweight fill option was selected. Elastizell was used to keep horizontal stresses against the bridge abutment to a minimum. The Michigan Department of State Highways and Transportation covered the foamed concrete with approximately 1.2 m (4 ft) of sand subbase to reduce the number of freeze-thaw cycles to which the concrete might be exposed.

The construction report for this project concluded:

1. Placement of the foamed concrete by the producer and its equipment was easy with relatively few construction problems. Due to the low viscosity, it was recommended that any transverse slopes be step tapered using basic concrete forming procedures.
2. Specifications must be developed to enable acceptance on the basis of 2- or 3-day compressive strength and dry density.
3. Due to material property characteristics (cost, low compressive strength, nonhomogeneity, water sorption characteristics, and equilibrium wet density), Elastizell

concrete cannot be recommended for general use as a lightweight fill material without qualifications.

A follow-up study had not been conducted at the time this paper was written.

Pine River Bridge, St. Clair, Michigan (13)

Another construction site was the Pine River Bridge project in St. Clair, Michigan (1976). The original bridge had been rebuilt in 1933 but had become increasingly difficult and costly to maintain. The area consisted of deep, soft alluvial clays that resulted in extensive settlement that required structural repairs. A new bridge was designed that included a 1.1-m (3.5-ft) raise in grade.

Because of the soft clay subgrade, two design alternatives were considered to prevent future settlement: the construction of additional approach spans or the use of a lightweight fill material. It was estimated that the use of a lightweight fill material could save as much as $200,000 for this project.

The technical report for this project contains conclusions similar to the Waiska River project in that the Elastizell was easily placed, although specifications and qualifications must be further developed.

Performance of Foamed Concrete

In a later report concerning the two Michigan projects mentioned previously (*14*), the Michigan DSHT concluded:

1. In general, Elastizell concrete appears to be a satisfactory lightweight fill material. The material has adequate strength, remains lighter than the design unit weight and does not absorb water, and settlements are negligible.
2. Although structurally sound, the Elastizell fills contain a large number of soft, powdery areas that have little or no support value. To date, these areas have not appeared to be damaging, but they should certainly be eliminated or minimized by better batch-mixing control.
3. No conclusions concerning the long-term performance of Elastizell were made at this time. Sampling and testing of the fills continue to monitor any changes in settlement, in-place density, or moisture content that may take place with continued exposure.

Findings

General

The current level of technology being applied to the design of roadways using lightweight fills is based on either experience or the adaptions of other design guidelines.

Although a material may have a high cost, an economic study should be conducted to determine the cost per kilogram load reduction per square meter to determine the total cost. A cost study may show that the more expensive material may require less excavation and therefore require less fill material compared to the less expensive lightweight fill material.

For example, a construction area located over soft alluvial clays requires a load reduction of the subgrade materials, 2440 kg/m^3 (500 lb/ft^2), to minimize future settlement. Assuming an in-place wet density of the surface materials to be 2002 kg/m^3 (125 lb/ft^3), the following excavations and fill depths would be required: shredded tires, 1.65 m; wood chips, 1.44 m; and polystyrene foam, 1.28 m.

A cost analysis would include the cost of the excavation, lightweight fill, and fill placement.

$$\$(\text{excavation}) + \$(\text{fill material}) + \$(\text{fill placement}) = \text{comparable fill cost}$$

Although shredded tires may be cheaper, polystyrene foam requires less excavation and is placed easily without the need of heavy equipment.

Sawdust, Wood Chips, and Corduroy

Wood material is viable as a lightweight fill; however, it is biodegradable. Design parameters are known and many agencies have experimented with wood materials. Some wood materials are considered a by-product and therefore are generally inexpensive.

Expanded Polystyrene

Although EPS has had limited use in Michigan and even less use in the rest of the United States, it has been used extensively in Norway and Finland where it is performing well. Further use of EPS in the United States may prove to be beneficial and cost-effective.

Waste Tires

Shredded tires are an effective lightweight material that may be used above the water table. They provide adequate strength and load transfer while reducing the static load on the underlying soils.

Currently the MPCA restricts the use of tires within a saturated zone. A composite alternative using waste tires has been to place wood chips to a depth of 0.3 m (1 ft) above the water table, taking care to prevent biodegradation, and continue the fill with waste tires. However, the use of one fill that is nonbiodegradable and inexpensive throughout all layers, in comparison to using numerous fills, could cut the cost of expanding our roads across load-restrictive soils.

Foamed Concrete

Foamed concrete is a high-strength, low-density material that would work well in many areas. The cost of foamed concrete is high; therefore, it may be justified only in special cases where high strength and limited horizontal forces are required.

REFERENCES

1. Nelson, D. S., and W. L. Allen, Jr. *Sawdust as Lightweight Fill Material.* Report FHWA-RD-74-502. FHWA, U.S. Department of Transportation, 1974.
2. Buss, K. G. Use of Sawdust on Forest Roads. *Proc., 35th Annual Road Builders' Clinic,* College of Engineering Extension Service, Washington State University, 1984.
3. Lukanen, E. O., and G. Teig. *Design and Evaluation of Roadway Widening Sections Through Swamps.* Inv. No. 199, Initial Report. Minnesota Department of Transportation, St. Paul, 1978.
4. Smith, R. S., and T. S. Coulter. Use of Lightweight Sawmill Residue in Highway Embankments. *RTAC Forum,* Vol. 3, No. 1, 1980, pp. 84–89.
5. Coleman, T. A. Polystyrene Foam Is Competitive, Lightweight Fill. *Civil Engineering,* Feb. 1974, pp. 68–69.
6. AabØe, R. 13 Years of Experience With Expanded Polystyrene as a Lightweight Fill Material in Road Embankments. *Veglaboratoriet.* Norwegian Road Research Laboratory, Oslo, Aug. 1987.
7. Drescher, A., D. Newcomb, and M. Bouhaja. *Development of Design Guidelines for Shredded Tire Fill in Road Subgrades and Retaining Walls.* Interim Report. Department of Civil and Mineral Engineering, University of Minnesota, 1991.
8. Ronchak, A. *Waste Tires in Sub-Grade Road Beds.* Minnesota Pollution Control Agency, St. Paul, 1990.
9. *Shredded Tires as Lightweight Fill Material.* Technical Fact Sheet. Minnesota Pollution Control Agency, St. Paul, 1990.
10. Ohrn, J. Lightweight Tire Rubber Fill. *The Center Line,* Winter 1990, p. 6.
11. Rudd, J. C., and B. Loney. *The Use of Shredded Tires as a Lightweight Subgrade on 9-Ton Road.* In-House Report. Prior Lake, Minn., 1991.
12. Novak, E. C., Jr. *Elastizell Concrete Lightweight Fill Construction.* Research Report No. R-1064. Michigan Department of State Highways and Transportation, Lansing, 1977.
13. Novak, E. C., Jr. *Experimental Lightweight Fill.* Research Report No. R-1053. Michigan Department of State Highways and Transportation, Lansing, 1977.
14. Green, T. M., and E. C. Novak, Jr. *Condition of "Elastizell" Lightweight Concrete Backfills After One Year in Place.* Research Report No. R-1096. Michigan Department of State Highway and Transportation, Lansing, 1978.

The opinions, findings, and conclusions in this paper are those of the authors and not necessarily those of the Transportation Research Board.

Methods for Repairing Frost Damage on Gravel Roads

Asko Pöyhönen, *Finnish National Road Administration*

In Finland, gravel roads are mainly "unbuilt," meaning that the road structure is thin or the structural courses have mixed with subgrade material. Because of this kind of structure and Finland's geographical location (freezing index), Finnish gravel roads suffer from considerable frost damage every spring. The objective of this project has been to determine technically feasible and economical methods for repairing different types of frost damage. The project can be divided into four parts, of which three—the test pit studies, industrial waste material analyses, and studies of 15 different types of test structures—have already been completed. Currently, a follow-up of the test structures is being performed. Test structures were erected in 1991–1992. The length of each structure was 50 m (55 yd). Test structure studies can be divided into stabilization, intensification of drainage, thermal insulation materials, and geogrids and strengthening fabrics. In the stabilization of structural courses, efforts were made to use industrial waste materials. For the follow-up on the functioning of different structural solutions, the structures were equipped with electrical measuring instruments with which it is possible to measure the temperature and moisture distribution in the subgrade and pavement of the road, vertical movements of the road surface, and the deformation behavior of the strengthening fabric. The effect of stabilization on the improvement of the bearing capacity of the road has been significant. The effect of stabilization in balancing the differences in frost heaving has been the same or slightly better than that of reinforcements. However, experience gained from drainage structures is not very promising.

The Finnish National Road Administration (FinnRA) is responsible for the maintenance and care of 77 890 km (48,380 mi) of public roads. Of this almost 39 percent are gravel roads. In 1993 FinnRA spent about 250 million markkaa (FIM) ($45 million U.S.) on the maintenance and repair of gravel roads (winter maintenance not included). Although a large number of all public roads are gravel, about 95 percent of the traffic on public roads occurs on paved roads. However, the share of maintenance costs for gravel roads is almost 20 percent.

Finnish gravel roads are mainly "unbuilt," meaning that the road structure is very thin or the structural courses have mixed with subgrade material. Because of this kind of structure and Finland's geographical location (freezing index), the gravel road network suffers from significant frost damage every spring. If the frost damage period is prolonged, weight limits are imposed during the thawing period. The usual weight limit is 8 tons (8.9 metric tons). Frost-damaged roads cause extra expense to both the road owner and the road users.

In 1990, a project for determining technically feasible economical methods for repairing different types of frost damage on gravel roads was initiated (*1*). The project can be divided into four parts, of which three—the test pit studies, material analyses (industrial waste materials), and studies of 15 different types of test structures—have already been completed. Currently a follow-up of the test structures on the roads rehabilitated with new materials or repairing methods is being performed.

Test Pit Studies

More than 50 test pits were dug to ascertain the main reasons for the different types of frost damage on gravel roads. One of the observations made is that the structural layers (if there are any) have mixed with subgrade materials. As a result of this change, the bearing capacity of the road has lowered. On several occasions, this mixed road structure has turned into frost-susceptible material. Other factors that have contributed to road damage include the thickness variation of structural courses in the cross direction of the road, the groundwater table or rock reaching close to the road surface, settlement of subgrade materials, inadequate drainage, and too steep inner slopes of side ditches (Figure 1). In addition to traditional field investigations, ground-penetrating radar was also used. The bearing capacity of the road was found to be only 20 to 30 MN/m^2 at its lowest. When designing the test structures, the target bearing capacity was set at 70 to 80 MN/m^2.

Industrial Waste Materials and Material Analyses

Waste Materials Selected

One of the objectives of the project was to use industrial waste materials (by-products) in repairs. If a feasible and economical use could be found for a waste material in the road structure, it not only would reduce waste but would also save natural gravel resources. The following waste materials were chosen for the study: biotite, dehydrated gypsum, blast-furnace slag, power station ash, and materials with the brand names Finnstabi and Lohjamix.

The average annual amount of gypsum produced during the processing of phosphoric acid is 1.4 million tons (1.5 × 10^6 metric tons) and that of biotite developed during the preparation of apatite is about 5 million tons (5.5 × 10^6 metric tons). The annual amount of blast-furnace slag produced during the manufacture of pig iron is about 450,000 tons (0.5 × 10^6 metric tons). Blast-furnace slag resembles sand in its grain size distribution but the specific surface area of blast-furnace slag is much greater. Fly ash is produced in various localities and has useful properties that include lightness and high thermal insulation capacity. Finnstabi is produced during titanium dioxide manufacture in connection with water neutralization. During the production process, substances that improve Finnstabi's stabilization characteristics are added to it. Lohjamix is a product that contains sulphuric gypsum recovered during the flue gas filtration mainly at coal-fired power stations.

Material Studies

One of the disadvantages of the waste materials is the lack of detailed knowledge about their properties. This is why material analyses of the waste materials selected for the study were considered essential. The properties analyzed in the laboratory included the type and quantity of the binding material needed with the waste material, freezing-thawing behavior, strength, solubility, thermal conductivity, frost susceptibility, and dynamic load-carrying capacity. A summary of material analyses is given in Table 1.

Test Structures

Test sections were constructed in 1991–1992. The length of each test section is 50 m (55 yd), and they are

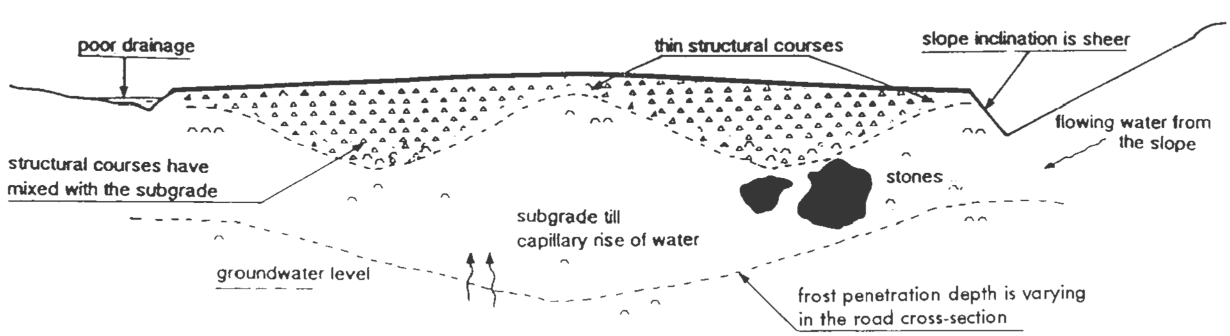


FIGURE 1 Some factors that can contribute to frost damage.

TABLE 1 Some Results of Waste Material Analyses

Material	Property				
	Strength, 30 days MPa	Strength, 90 days MPa	Dynamic modulus of deformation	Thermal conductivity W/mK	Permeability x 10^{-9} m/s
Crushed material + Finnstabi + CaO 8:1 16%	12	14	5,3	1,0	8,4
Crushed material+ blast-furnace slag 30% +CaO 0,6%	1,2	2,1	-	0,8	270
Crushed material+ Lohjamix 18%	5,3	10	0,55	1,0	1,8
Crushed material+ gypsum + cement 1:1 10%	13,5	17,5	-	0,7	-
Power station ash + blast-furnace cement 4%	5,2	11	-	-	<400

located in five maintenance areas. There are 15 different types of test sections. Each test section has a control section. No improvement or repair measures have been implemented in the control section. Consequently, we can estimate how well our test structures have improved the situation on the road.

Intensification of Drainage

By enhancing the drainage of structural courses of the road, it is possible to directly affect the bearing capacity and frost-heaving behavior of the road. As the water content of structural courses decreases, frost action is reduced and the bearing capacity of the road increases. For improving the drainage at damaged sites, underdrains and horizontally or vertically mounted hydraway drains were employed.

A hydraway drain (Enkadrain P32) was placed in the vertical position on both verges of the road (Figure 2). Excavation work had to be performed with a narrow shovel to keep the quantity of excavated and filled ground as small as possible. The trench was filled with sand, and the water collected by the hydraway drain was carried away from the structure through the drainpipe in the lower part of the drain.

In one test structure (Figure 3) a V-shaped hydraway drain (Filtram 1B1) was laid under the road structure so that, in the center of the road, it was at a depth of about 80 cm (2.6 ft) and on the edges, at a depth of about 40 cm (1.3 ft) from the road surface. In the cross section, the angle of gradient of the hydraway drain is 1:10. After the hydraway drain had been placed, the old materials of the structural courses were reused on the drain. The hydraway drain not only dries structural layers of the road, it also reduces the capillary rise of water into the road structure.

Reinforcing Fabrics and Geogrids

The purpose of reinforcements is to transfer the tensile stresses acting on the structural courses to the reinforcements to improve the strength properties of the road structure. So far, reinforcements have not been used on gravel roads partly because of their high price and partly because their advantages have not been realized.

Theoretically, the use of reinforcements leads to thinner structural courses than the use of filtration geotextile alone (Figure 4). Both geogrids (Fortrac 35/20-20) and strengthening fabrics (Televev 150/150) were used in the structural layers. Nearly all test structures in which reinforcements were used were situated in weak soil areas. Where possible, old structural layer material was used in the test structures. In one test structure, the edges of the road were strengthened with "bags" made of strengthening fabrics (Figure 5).

Stabilization

In the stabilization of structural courses, efforts were made to use industrial waste materials. Industrial waste materials often have properties that are needed in earthworks. Waste materials strengthen either by themselves or with an aggregate, in addition to which some waste materials have a high thermal resistance. Stabilization mainly brings the same benefits as reinforcements. A stabilized layer forms a flat sheetlike structure. It not only improves the bearing capacity of the road but also evens out irregular frost heaving and prevents layers from mixing with subgrade material. The purpose of stabilization is to find structures with as high a deformation resistance as possible. When this technique is applied, materials of the old road structure can be used efficiently. It was necessary to use binders with almost every waste material. The binding materials used with the waste materials and the binder contents are shown in Table 1. The objective was to make the stabilized layer, through its strength value, not as rigid as possible but as flexible as possible. The thickness of the stabilized layer was 15 cm (6 in.).

Thermal Insulation Materials

The research includes two solutions for thermal insulation. Compared with average costs, expanded clay is

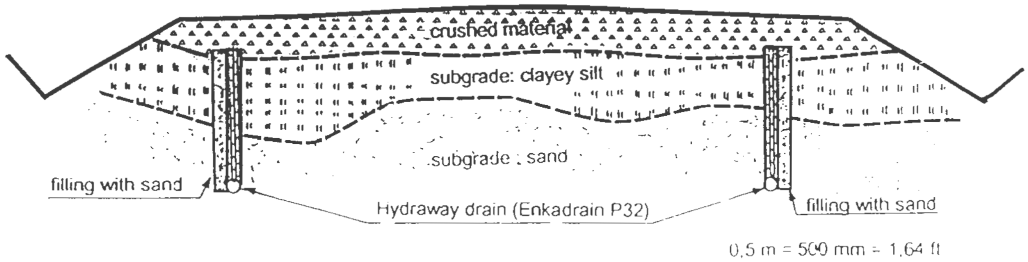


FIGURE 2 **Vertically mounted hydraway drain (Enkadrain P32).**

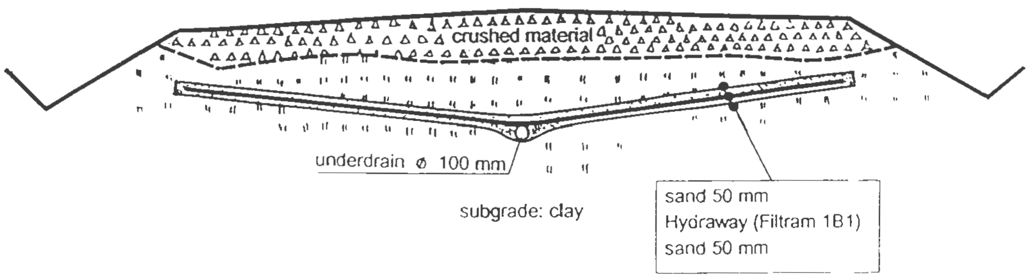


FIGURE 3 **Horizontally mounted hydraway drain (Filtram 1B1).**

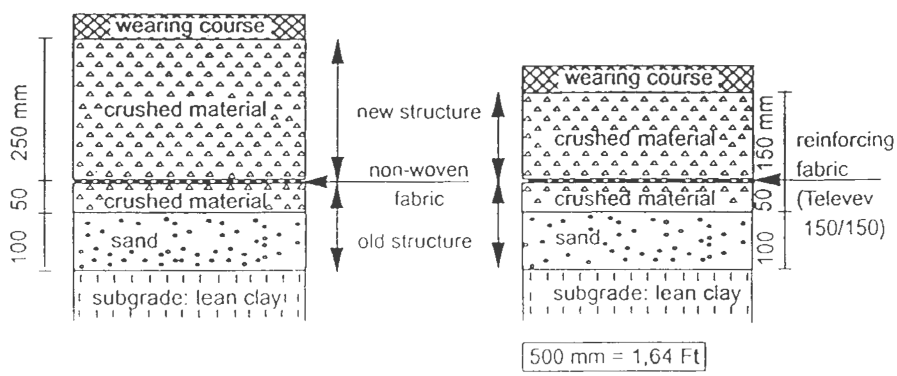


FIGURE 4 **Comparison of use of filtration geotextile and reinforcing fabric.**

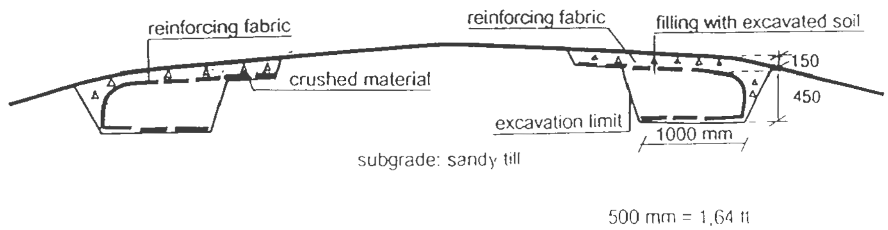


FIGURE 5 **Edges of road supported by strengthening fabrics.**

a considerably expensive solution. On the other hand, in weak soil areas expanded clay has not only a thermal insulation capacity but also a lightening effect on the structure. An insulation layer made of power station ash is a significantly cheaper alternative. The thickness of both layers was 20 cm (7.9 in.).

Realization of Test Structures

Of the constructed structures, those where the new structure lies directly on the old structure were clearly easiest and quickest to construct. The addition of a new layer of crushed stones and the possible use of reinforcements (usually, a filtration geotextile) are the traditional methods for repair because they are feasible and simple to use.

The use of industrial waste materials, even in separate layers, may be an alternative worth attention if waste materials are located close to the place where they can be used. Stabilization with waste materials will obviously be a viable alternative for improving the bearing capacity of the road. The present problem is mainly to find suitable working methods (spreading and mixing). Furthermore, long transportation distances of the waste materials used as binders may be an obstacle to their use. The advantage of stabilization is that the old road structure can be used. Solutions related to the enhancement of drainage are usually more difficult to realize. When enhancement of drainage is planned, it should be ascertained that the water collected by drainage structures can be carried away from the road area at moderate costs.

Follow-Up and Functionality of Test Structures

Follow-Up

To follow up on the functioning of different structural solutions, the test structures were equipped with electrical measuring instruments. These electrical instruments measure the temperature and moisture distribution in the subgrade and pavement of the road, vertical movements (frost heaving) of the road surface, and, in one test structure, the deformation behavior of the geogrid. Instrumentation was conducted as shown in Figure 6. The control sections were also instrumented.

Once a year, the groundwater and surface water of the structures in which industrial waste materials have been used are tested for heavy metals. The strength development of the stabilized layers is followed using test specimens taken from the structures. Their strengths are determined in the laboratory. The bearing capacity of the test structures and the control sections are measured each spring.

Functionality

Since the experimental structures were erected only in 1991–1992, no definite conclusions can be drawn about their performance. Since the economy of a structure depends on its lifespan, no conclusions can be drawn about economy at this stage. Frost heaves in the control sections have been as much as 40 cm (15 in.) at their highest and the frost depths about 2 m (6.5 ft).

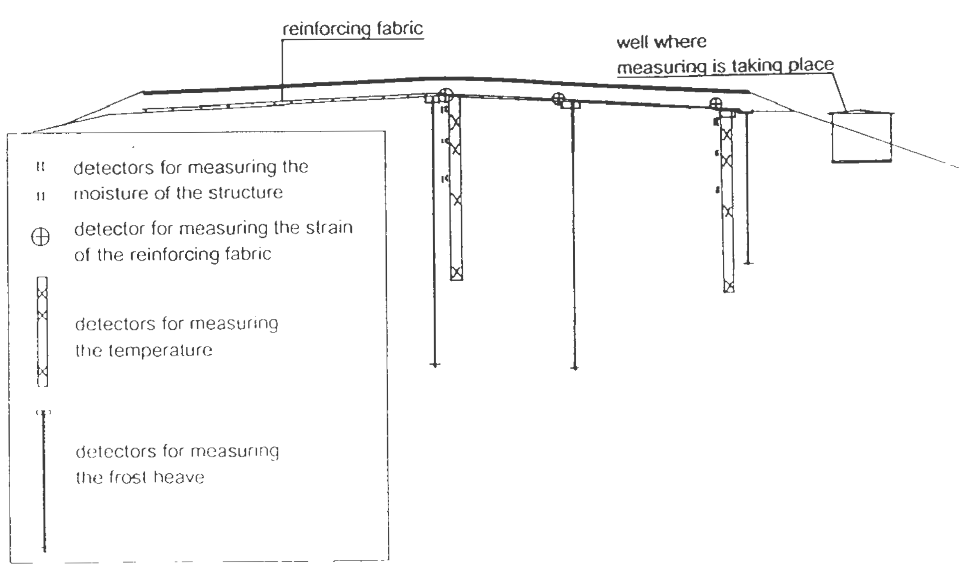


FIGURE 6 Instrumentation of test structures.

Knowledge gained from drainage structures is not promising. The construction of these structures is more expensive and difficult than on average. Their function has been unreliable and the results achieved from improving the bearing capacity and balancing the movements caused by frost have been worse than in other test structures on average.

By using geogrids or strengthening fabrics, it is possible to reduce the thickness of the crushed stone layer needed in reparation construction and, consequently, to save on material and transportation costs. According to the strain measurements, reinforcements have been functioning as expected (i.e., they have strengthened the road structure). Reinforcements have significantly balanced the uneven frost heaving [i.e., the differences in frost heaving have dropped to half of what they used to be within 5 m (5.5 yd)].

The effect of stabilization on the improvement of the bearing capacity of the road has been significant. At its best, the bearing capacity has improved by 50 percent as a result of stabilization. The effect of stabilization in reducing the differences in frost heaving has been at the same level or slightly better compared with that of reinforcements.

Extremely good results were achieved from the use of power station ash. When properly compacted, ash forms a flat sheetlike structure that improves the bearing capacity of the road and serves as a thermal insulation material. After construction, frost heaves have reduced to half of the previous level and the bearing capacity of the road has more than doubled. The expanded clay structure has behaved as expected with the exception of the settlement that has occurred.

REFERENCE

1. *Methods for Repairing Frost Damages on Gravel Roads.* FinnRA Reports 27/1991, 32/1992, 50/1992, and 52/1992 (in Finnish). Finnish National Road Administration, Helsinki.

Transforming a Railroad Grade into a Multiuse Transportation Corridor

Merv Eriksson and Jim Penzkover, *USDA Forest Service, Missoula, Montana*

The Chicago, Milwaukee and St. Paul Railway Company constructed its westward extension through Missoula, Montana, to Tacoma, Washington, in 1907 and 1908. Design, construction, and maintenance of the 72-km (45-mi) section of railroad grade from St. Regis, Montana, to Avery, Idaho, involved engineering feats that have valuable historical significance. Fills were placed under existing temporary trestles using hydraulic mining technology. Tunnels were dug using air-powered shovels and electric ore cars. The regenerative electrification of the line through western Montana to Avery, Idaho, in 1917, was the first major railroad electrification in the world. Preserving the history of these engineering feats is a worthwhile challenge. After the Milwaukee Railroad abandoned the line in 1980, engineers with the USDA Forest Service, the Federal Highway Administration, and Shoshone County in Idaho began evaluating the route for various transportation needs. In 1988 the Forest Service and Shoshone County cooperated in reconstructing a segment of the railroad grade into a county road. This 14.5-km (9-mi) segment with six tunnels and two large trestles cost about $727,000 to reconstruct. The $50,000/km ($81,000/mi) reconstruction cost was far less than the cost of new construction through this difficult country. In 1991 the Federal Highway Administration reconstructed a 21-km (13-mi) railroad segment from Avery, Idaho, west, as Forest Highway 50. The flat, straight alignment of the railroad grade was conducive to a very economical, high-speed reconstructed roadway. Since 1993, the USDA Forest Service has been evaluating and designing repairs and modifications to allow pedestrian and mountain biker use of the 58-km (36-mi) segment from St. Regis, Montana, westward. Repair and reconstruction of this segment would include preservation and interpretation of the historic engineering features of the route. This segment includes 11 tunnels [the longest of which is 2666 m (8,771 ft)] and 9 large trestles.

Various sections of the abandoned Milwaukee Railroad in western Montana and northeastern Idaho are being converted to multiple-use transportation facilities by the USDA Forest Service, the Federal Highway Administration, and Idaho's Shoshone County.

Construction of the Chicago, Milwaukee and St. Paul Railway from Missoula, Montana, through Avery, Idaho, began in 1907 and was opened to train traffic in 1908 (*1*). This segment of railroad line [a distance of 121 km (75 mi)] follows the Clark Fork River, from Missoula to St. Regis, Montana, where it begins its climb over the Bitterroot Mountains (Figure 1). From an elevation of 760 m (2,500 ft) at St. Regis, the railroad grade climbs to an elevation of 1268 m (4,170 ft), 35 km (22 mi) later at the Montana-Idaho border. This elevation is actually 305 m (1,002 ft) below the ground surface. The railroad enters a 2666-m-long (8,771 ft)

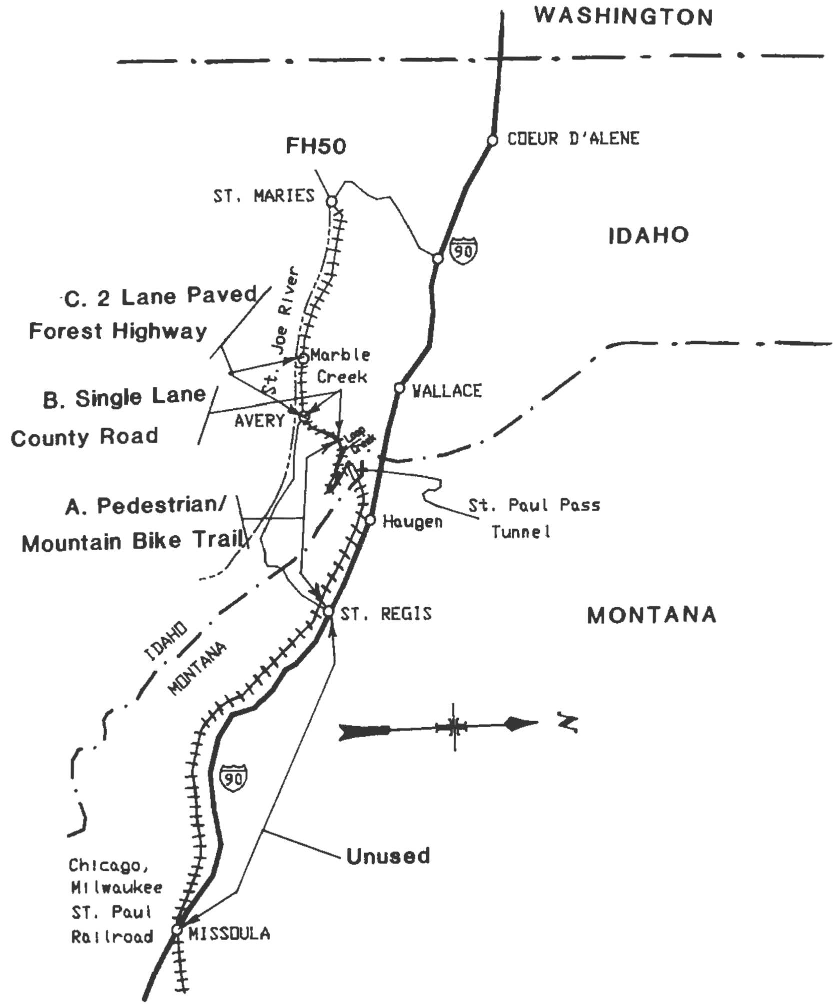


FIGURE 1 Area map.

tunnel that made maintaining a maximum 1.7 percent grade possible through this steep rugged country. The railroad grade drops back to a 730-m (2400-ft) elevation, 37 km (23 mi) later at Avery, Idaho. From there, the railroad grade follows the scenic St. Joe River to St. Maries and, eventually, Tacoma, Washington. From St. Regis, Montana, to Avery, Idaho, the railroad crossed 18 bridges, 11 of which are major trestles, and passes through 17 tunnels. The trestles rise over 76 m (250 ft) out of canyon bottoms and vary in length up to 258 m (850 ft). The 17 curved and straight tunnels vary in length from 54 m (178 ft) to 2666 m (8,771 ft).

History

The rugged Bitterroot Mountain Range with its steep canyons posed an almost insurmountable obstacle to the construction of the original Chicago, Milwaukee and St. Paul Railway from Missoula, Montana, westward in 1907 and 1908. The final 305 m (1002 ft) of climb over the Bitterroot Mountains was avoided by construction of the St. Paul Pass Tunnel (2).

Construction of the tunnel was an engineering feat of amazing proportion (Figure 2). Electric power was available at both portals in these remote mountain outposts when New York and Chicago were just beginning to experience electrification. Air shovels were used to load materials on electric cars. The bore pushed through at a rate of an average of 6 m (20 ft) per day from both ends for about 7 months. Both crews met in the center of this 2666-m (8,771-ft) tunnel only 3 to 5 cm (1 to 2 in.) off-line. The tunnel is straight with a 0.2 percent grade rising from both portals to a high point in the center. It is 5 m (16.5 ft) wide by 7 m (23 ft) high at the center of the arched roof. The tunnel has treated timber snowsheds at both portals and full concrete lining extending for most of its length. The remainder of

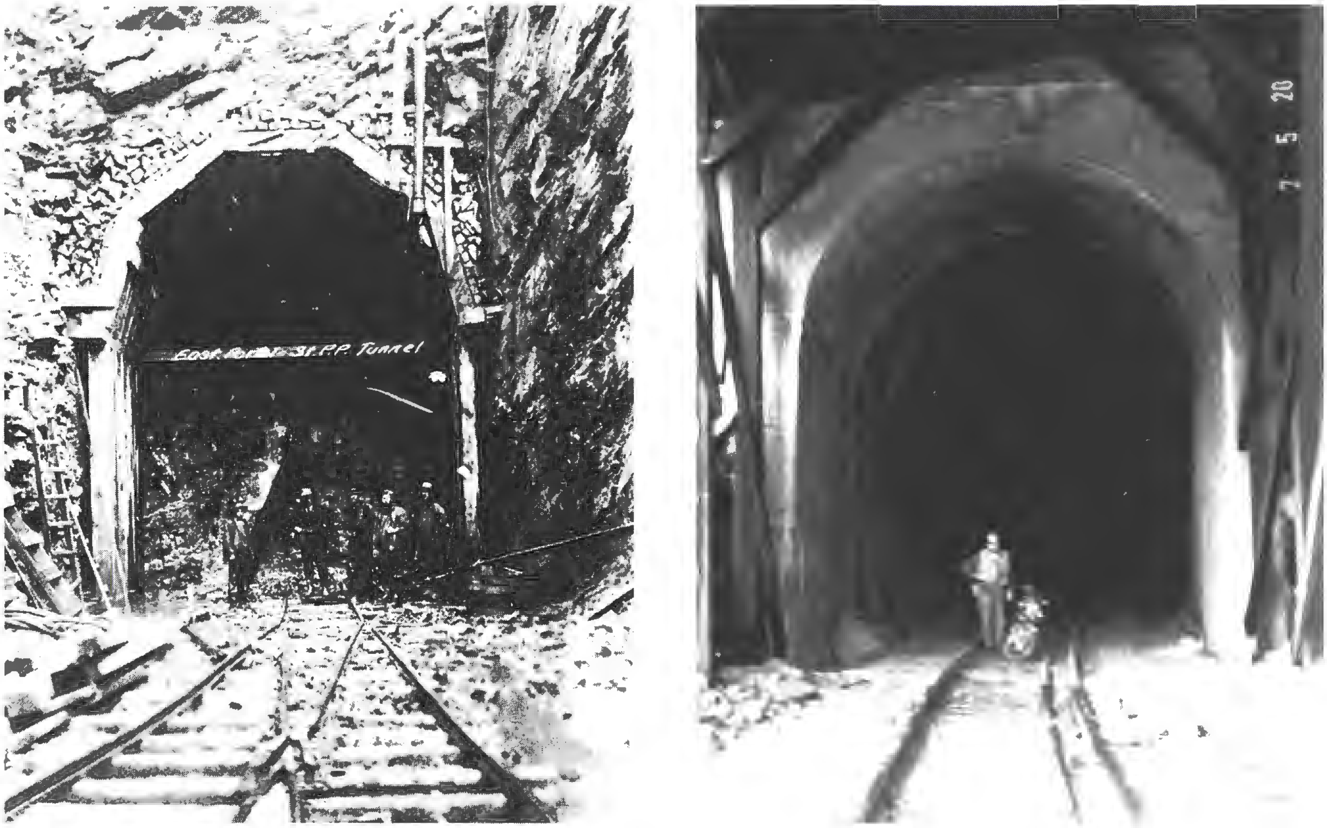


FIGURE 2 ***Left*****, east portal, 1907;** ***right*****, east portal, 1982.**

the tunnel is partially lined with concrete and timbers. The unlined portions are in quite competent rock. As expected of such a long bore over 300 m below a mountain pass, a sizable stream of water runs out both tunnel portals.

The $46,600/km ($75,000/mi) construction cost in 1907 dollars for the railroad grade from the St. Paul Pass Tunnel to Avery, Idaho, was the most expensive segment of railroad in the world in 1907 (*3*). The 10 tunnels in this section of railroad were lined with 4720 cm^3 (2 million bd ft) of timber. The seven trestles had a combined length of over 1200 m (4,000 ft).

The original 1908 railroad startup with steam and coal-fired locomotives lasted only 2 years. In 1910 devastating wildfires burned most of northern Idaho, including most of the wooden trestles on this route. Milwaukee trains saved whole communities from the fires by transporting them to the tunnels near the Idaho border (*4*). The burned trestles were reconstructed with steel trestles or earth fills (Figure 3).

In 1914 the Chicago, Milwaukee and St. Paul Railway started converting to electrified engines; by 1917 all infrastructure was in place and operating on the 610 km (380 mi) from Harlowtown, Montana, to Avery, Idaho (*5*). Electrical substations were constructed every 61 km (38 mi). The electric locomotives used "regenerative braking" when descending mountainous grades. They generated 40 to 60 percent of the power needed for the next train coming up the grade (*6*).

The railroad operated until 1980, when the Milwaukee Railroad officially closed its lines west of Miles City, Montana. In the early 1980s, as the track and rail were being salvaged, the potential value of portions of this grade as a transportation corridor was becoming evident (*7*).

Most of the abandoned railroad grade from Missoula to St. Regis, Montana, is currently unused. Much of the right-of-way is privately owned, and the track and ties have been salvaged and much of the grade obliterated.

Reconstruction of Route

The railroad grade from St. Regis through the 2666-m-long (8,771 ft) St. Paul Pass Tunnel to the confluence of Loop Creek and the North Fork of the St. Joe River is presently being evaluated for use as a mountain bike and hiking trail. Design work is under way to repair the

FIGURE 3 Temporary trestle during fill placement.

tunnels and trestles, to erect interpretive displays, and to construct parking and tourist facilities.

The railroad grade from the mouth of Loop Creek to Avery, Idaho [14.5 km (9 mi) following the North Fork of the St. Joe River] has been converted to a single-lane county road. This stretch includes six tunnels and two major trestles.

The line from Avery, Idaho, west to Marble Creek was converted to a two-lane paved forest highway. This 21-km (13-mi) project was designed and constructed by the Federal Highway Administration (*8*).

Following is a discussion of (a) the 58-km (36-mi) proposed trail segment from St. Regis, Montana, to the mouth of Loop Creek in Idaho; (b) the 14.5-km (9-mi) segment that was converted to a single-lane county road, from the mouth of Loop Creek to Avery, Idaho; and (c) the 21-km (13-mi) segment that was converted to a double-lane forest highway.

Trail Segment

The 58-km (36-mi) trail segment of railroad grade is truly one of the engineering marvels of this century. The grade parallels the St. Regis River for 26 km (16 mi) before beginning its final assault on the Bitterroot Mountains. The railroad grade then climbs 8 km (5 mi) to the east portal of the St. Paul Pass Tunnel. After exiting the tunnel, the railroad grade makes a 23-km (14-mi) switchback down to the North Fork of the St. Joe River. The impressive trestles, massive rock bore tunnels, towering earth fills, and the electrification of the entire segment bear witness to the engineering creativity and determination of the builders. Preservation of the remaining features is historically vital and could provide an outstanding recreational opportunity.

The Milwaukee Railroad struggled financially for many years and although the quality of the initial construction was amazingly good, minimal maintenance after 1970 has caused deterioration. In 1982, a company purchased salvage rights to the rails, ties, copper wire, and timber wire support towers. Little regard was given to maintaining drainage or the historical nature of any of the system. In 1986, all private landholdings along the railroad grade and all remaining aboveground construction features, from the St. Paul Pass Tunnel to Avery, Idaho, were purchased by the federal government, using money provided by a special act of Congress (*9*).

Most of the ties and all of the rail were removed from the grade, but in the St. Paul Pass Tunnel the ties and much of the ballast were bulldozed to either side to allow trucks from the salvage company to haul through the tunnel. The drainage ditches were blocked and perforated drainpipe was torn up. In 1987, a U.S. Army Reserve engineering unit spent its 2-week summer training time regrading and cleaning out the tunnel. The Forest Service, in 1991, evaluated converting the St. Paul Pass Tunnel to a logging truck haul route. The ideal was dropped due to the high cost of repair and potential problems with one-way hauls through a tunnel this long. Meanwhile, the public was discovering this natural transportation corridor; use of the grade by mountain bikers and pedestrians was increasing steadily. In 1993, because of liability concerns, the Forest Service closed the St. Paul Pass Tunnel to all traffic. Concrete closures, with small steel doors for administrative access, were installed at each portal.

Although the tunnel is closed to the public, the railroad grade, from both ends of the tunnel, continues to be very popular. These 58 km (38 mi) of railroad grade traverse some of the most spectacularly beautiful country in North America. In 1993, the Forest Service, at the urging of a local economic development group, began studying the feasibility of converting the entire route to a trail system. The maximum vertical gradient of the route is 1.7 percent, which is ideal for mountain bike and handicapped use. The ballast varies from 0.6 m (2 ft) deep in the tunnels to over 30 m (100 ft) deep in some of the larger fills. The ballast itself varies from crushed rock to gravels hydraulically washed off adjacent hillsides to stream-washed rock (*10*). In some of the areas where course or loose rock is present, finer material or surfacing will be needed for a stable riding surface. Most of the culverts under the fills, or "water tunnels" as the railroad referred to them, are reinforced concrete boxes. They are in relatively good condition and require only minor cleaning and repair.

The 11 large trestles are in relatively good structural condition (Figure 4). All trestles have reinforced concrete spread footings supporting steel lattice piers. Two built-up steel beams 2.3 m (7.5 ft) apart support the deck system across the spans, which vary in length from 17.6 to 22.8 m (58 to 75 ft). The decks are either reinforced concrete or treated timber "boxes" filled with rock or gravel ballast. The abutments are reinforced concrete. One-meter-wide timber walkways are cantilevered from the deck systems and, in places, a two-cable handrail system is still in place.

The concrete is spalling on many of the abutments, but all high-stress-bearing areas are in good condition. Some light surface rusting is occurring on the steel lattice, or "trestle" pier towers, and on the steel beams. The steel in these towers and beams would have to lose a large percentage of their cross section before being in danger of failing from dead loads, which are their only major loading after the trains quit running. Some of the concrete and treated timber ballast boxes have deteriorated and may have to be repaired or replaced; however, most are adequate to carry their ballast load for many years. The cantilevered timber walkways and rail systems are the major items needing repair before the trestles will be safe for pedestrian use (Figure 5). All deteriorated timber and the entire railing system will be replaced. The new railing will be pedestrian height [1.07 m (42 in.)] since bicycles will not be allowed on the walkways. The ballast will be lowered and regraded so the edge of the ballast boxes will act as a 4-in.-high curb.

The 16 shorter tunnels [the longest of which is 456 m (1,500 ft)] are in relatively good condition. In several areas, the concrete lining is spalling and may require repair. If the St. Paul Pass Tunnel is reopened, its repair will be the largest challenge. All tunnels are 7 m (23 ft) high and 5 m (16.5 ft) wide. The top third of the tunnel is a circular arch. The tunnels vary from being completely concrete lined to unlined. The St. Paul Pass Tunnel is fully or partially concrete lined throughout its length. About 2280 m (7,500 ft) of the tunnel has a full perimeter lining. About 395 m (1,300 ft) of the tunnel has a partial lining consisting of an arched roof, longitudinal beams at the spring lines, and columns on grade beams supporting the roof and longitudinal beams. The lining thickness is estimated to be 0.6 to 1.2 m (2 to 4 ft) in most of the tunnel. In general, the concrete lining is in very good condition. However, about 10 percent of the total length of the St. Paul Pass Tunnel is spalling, due primarily to groundwater leaching of cement from the concrete. Spalling areas have again plugged the drainage system, and water has ponded to a depth of 10 to 20 cm (4 to 8 in.) in several areas of the tunnel.

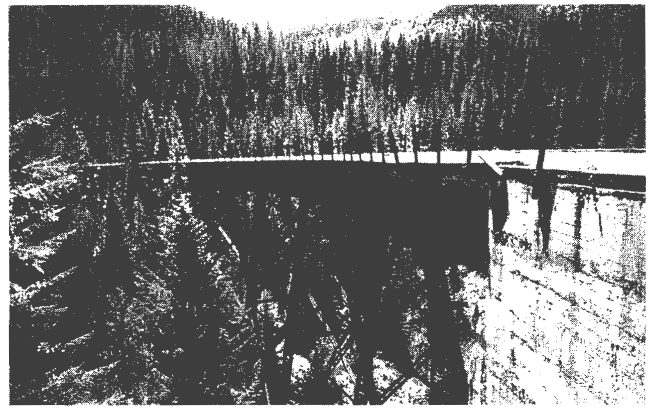


FIGURE 4 Dominion Creek trestle in Montana.

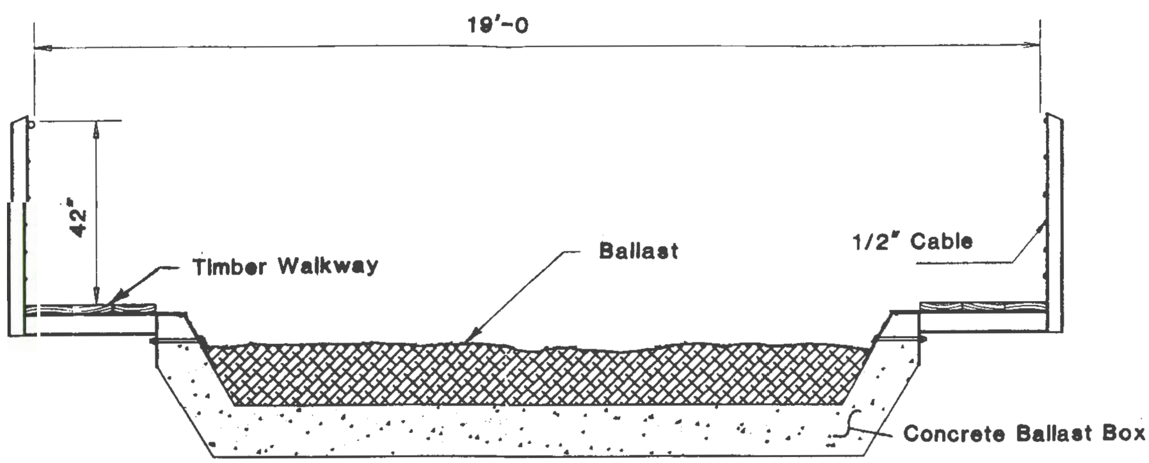


FIGURE 5 Proposed typical section for trestles.

The minimum repair necessary to make the tunnel usable by mountain bike and pedestrian traffic will be repair of the concrete liner, repair of the groundwater drainage system through the concrete lining, cleaning and regrading of the surface drainage system, and surfacing of the travelway surface. Estimates of the repair cost for just this tunnel vary upward from $1 million.

Arranging funding for repairs, interpretive displays, and other facilities is being pursued by the local economic development group. Private grants or contributions, Rails-to-Trails participation, and local government partnerships with the Forest Service are all possibilities.

County Road Segment

The county road from the mouth of Loop Creek along the North Fork of the St. Joe River to Avery, Idaho, was a narrow, substandard dirt road that could not accommodate logging trucks safely. Both the USDA Forest Service and Shoshone County officials recognized that the railroad grade, located on the other side of the river, had a far better alignment. In 1987, the Forest Service and Shoshone County entered into a cooperative agreement to convert the railroad grade to a county road, spelling out responsibilities for each party (*11*). This 14.5-km (9-mi) segment of the railroad grade has six tunnels [varying in length from 101 m (332 ft) to 194 m (638 ft)] and two trestles [72 m (238 ft) long and 157 m (515 ft) long]. The Forest Service agreed to reconstruct the trestles to 4.3-m-wide (14 ft) travelway, single-lane bridges with guardrail and approach rail meeting the static structural requirements of the American Association of State Highway and Transportation Officials. Shoshone County would resurface the new roadway and construct connections to existing roads.

In 1988, while the Forest Service reconstructed the two trestles, Shoshone County improved the surfacing by adding fines to the coarse aggregate railroad ballast. The county also signed the tunnels for safety, constructed the road connections, and removed sections of electric wire, old railroad ties, poles, and trash left behind from the many train derailments. A large electromagnet was rented and towed behind a pickup truck over the entire grade to remove railroad spikes and shards of steel. The county's portion of the reconstruction work cost approximately $250,000.

The curved trestle crossing Dick Creek is 72 m (238 ft) long. The straight trestle crossing the North Fork of the St. Joe River near Squaw Creek is 157 m (515 ft) long. Both trestles were originally constructed with reinforced concrete ballast boxes. These boxes were 4.3 (14 ft) wide (trestle width) and about 1.2 m (4 ft) long and held about a 30-cm (12-in.) depth of gravel ballast.

The Forest Service, through a contractor, removed the ballast and ballast boxes on the trestles and replaced them with match-cast concrete panels about 5.5 m (18 ft) wide and 3.2 m (10.5 ft) long. The concrete panels were prestressed in the 5.5-m (18-ft) direction (transverse to the trestle centerline) and posttensioned with high strength Dywidag bars in the longitudinal or 3.2-m (10.5-ft) direction. The concrete replacement panels are 30.5 cm (12 in.) thick at the bridge centerline and 25.4 cm (10 in.) thick at the edges of the deck. Prior to the placement of the new deck panels, the top surfaces of the built-up steel beams were cleaned and repainted. The new concrete panels are attached to the steel beams by bolted connections into threaded inserts, embedded in the concrete panels.

The contractor first removed the ballast from the boxes by bulldozing from the center of the trestle to the ends. The ballast boxes were then removed, beginning at the center of the bridge and working toward each end (the empty ballast boxes were later used to construct a retaining wall for one of the connector roads) (Figure 6). The new concrete deck was then installed beginning at one end of the trestle. Each panel had been cast against its adjoining members so the fit was nearly perfect. A bonding agent was applied to the surfaces between the panels before applying the jacking force that posttensioned the panels together. Each panel was posttensioned to its adjoining panel before the next panel was installed. The compressive force holding each panel together is greater than 845 kN (190,000 lb). Expansion devices were installed in the Squaw Creek Bridge deck to take the calculated 7.6 cm (3 in.) of temperature change movement.

After the new concrete deck was installed, a new structural tube vehicle guardrail system was installed. A pedestrian-height [1.1 m (42 in.) above the deck] pipe

FIGURE 6 Removal of first ballast box from Dick Creek trestle.

rail was mounted above the vehicle rail in anticipation of pedestrian use of the bridge.

All deteriorated concrete in the abutments was removed and replaced, and the expansion devices were replaced.

These trestles were converted to vehicle bridges at a cost of $634 per linear foot, for a total cost of about $477,000. The cost of this 14.5-km (9-mi) segment was about $727,000. This converted section of railroad bed is adequate for a single-lane roadway. Grades vary from 0.2 percent to 1.7 percent and the horizontal curves are very flat. The entire railroad grade has from 0.6 to 1.8 m (2 to 6 ft) of ballast rock, which required only a thin surfacing course to provide an excellent aggregate roadway surface. About one-half of the old county road on the opposite side of the canyon was converted to a trail. The remainder of the old county road was left open and accesses a campground, a trailhead, and a logging road.

Forest Highway Segment

In 1991, the segment of railroad grade from Avery, Idaho, to Marble Creek was opened for automobile and truck travel. This segment of railroad grade follows the St. Joe River, a river designated as wild and scenic. This 21-km (13-mi) segment is a double-lane, asphalt-paved, 80-kph (50-mph) forest highway (FH-50). The Forest Highway Program is a federal program that returns a portion of the federal gas tax revenue to the states for use on selected public roads that serve a substantial amount of national forest–related traffic and resources. In Idaho, the program is jointly administered by the Federal Highway Administration, the USDA Forest Service, and the Idaho Transportation Department. This forest highway project was engineered and administered by the Western Federal Lands Highway Division of the Federal Highway Administration. When construction of the project was completed, the highway was turned over to the county for maintenance.

The St. Joe River Road between Avery and St. Maries, Idaho, was placed in the Idaho Forest System in 1957. FH-50 serves a variety of important needs and uses within the St. Joe valley, including access to local residences, transportation of timber products, and recreational travel, as well as linking the small communities of St. Maries, Calder, Marble Creek, Hoyt Flat, Avery, and St. Regis. It is also a strategic arterial road for transporting logs from the forests of the large St. Joe drainage. Before 1991, the route from Avery to Marble Creek was a hazardous, single-lane, unsurfaced road with turnouts. It had poor alignment with inadequate sight distances, and its deteriorated condition required a high level of maintenance by the county. When the Milwaukee Railroad closed in 1980, the bankruptcy trustee sold the segment of railroad grade from Avery to St. Maries to Potlatch Corporation (a logging company). Potlatch used it for several years as a private short line for the transport of timber.

When the Federal Highway Administration began design work for the reconstruction of FH-50, the railroad grade was studied and selected instead of trying to improve the existing south side road through some very unstable slide areas. The railroad grade provided a better alignment in more stable terrain and resulted in reduced environmental impacts at a substantial savings in construction costs. The railroad grade, with its straight alignment and thick ballast, was the ideal location for FH-50. In about 1987, the Federal Highway Administration purchased this segment of the railroad grade from Potlatch Corporation for conversion to a vehicle travelway.

Construction costs for this 21-km (13-mi) segment were approximately $12.3 million, plus $1 million for rights-of-way purchase. A large portion of this cost was for the construction of two major bridges: one across the St. Joe River, near Marble Creek, and the other across the North Fork of the St. Joe River, at Avery. The original railroad bed was a minimum of 4.3 m (14 ft) wide. The minimum curve radius was 243 m (800 ft) and the maximum gradient was 1.7 percent. The new forest highway has 2- to 3-m-wide (10 ft) lanes and 0.6-m (2-ft) shoulders. The reconstruction of the forest highway basically consisted of widening, paving, and installing guardrail.

Reconstruction of the previously existing county road on the other side of the St. Joe River, to the same design standards, would have cost approximately $25 million.

Summary

The civil engineering feats on this portion of the Milwaukee Railroad are comparable with any in North America over the past 90 years. Eleven major trestles with a total length over 1.9 km (1.2 mi) rise high over steep canyon bottoms. Seventeen tunnels bore through 5.8 km (3.6 mi) of granite. Huge earthen fills, water tunnels carved through solid rock, and electrification of the railroad engines provide an exciting history of applied engineering skills. The 90-year history is as colorful as the surrounding mountains.

The past 14 years have been an exciting time for the local engineers who converted railroad grades, tunnels, and bridges for automobile and pedestrian use. The first 6 years were spent in controversy over ownership of the route as salvage companies ravaged the old railroad, tearing out rails, ties, and historical remnants of the railroad era. Since 1986, about one-third of the 93 km

(58 mi) from St. Regis, Montana, to Marble Creek, west of Avery, Idaho, has been converted to vehicle roadways. The remainder is in the planning stage for what could become a very popular trail system if construction money is available.

Tomorrow's challenges will be to preserve what remains for future generations to enjoy while sharing the legacy of the railroad era. The feats of the engineers and thousands of workers from all over the world who contributed to the construction of this engineering marvel should be preserved. The future holds endless possibilities for sharing this rich engineering heritage with the public.

REFERENCES

1. McCarter, S. *Guide to the Milwaukee Road in Montana.* Montana Historical Press, 1992.
2. Derleth, A. *The Milwaukee Road. Its First Hundred Years.* Creative Age Press, 1948.
3. Wood, C. R., and D. M. Wood. *Milwaukee Road West.* Superior Publishing Company, 1972.
4. Koch, E. *When the Mountains Roared: Stories of the 1910 Fire.* Manuscript on File. Idaho Panhandle National Forest, Coeur d'Alene, 1914.
5. Farrington, S. K., Jr. *Railroading from the Head End.* Doubleday, Dorin and Company, 1943.
6. *The Abandonement of Electric Operation by the Chicago, Milwaukee, St. Paul and Pacific Railroad Company.* Mimeographed. Northwest Rail Improvement Committee, Seattle, Wash., 1978.
7. Penzkover, J. *Transportation Summary Report: Avery, Idaho to I-90 at Taft, Montana Utilizing the Abandoned Chicago, Milwaukee, St. Paul and Pacific Railroad.* Idaho Panhandle National Forest, Coeur D'Alene, 1984.
8. *St. Joe River Road Reconstruction.* FHWA, U.S. Department of Transportation, Vancouver, Wash., 1991.
9. Pub. L. 99-190, 1987.
10. Sims, C. *Heritage Resources Specialist Report.* Idaho Panhandle National Forest, Coeur d'Alene, 1994.
11. Memorandum of Understanding. USDA Forest Service–Shoshone County. Idaho Panhandle National Forest, Coeur d'Alene, 1987.

Design, Construction, Maintenance, and Performance of Qinghai-Tibet Plateau Highway

Li Ningyuan and Ralph Haas, *University of Waterloo, Canada*

Qinghai-Tibet highway, the highway with the highest elevation in the world, was originally built in the 1950s without detailed design. It is about 860 km in length and goes through the center part of the Qinghai-Tibet Plateau with an average elevation of 4100 m. A number of permafrost engineering problems have been experienced since an asphalt surface layer was placed over the entire highway during the mid-1970s. Typical pavement damage and distress included large amounts of thaw settlement of embankments, unequal deformations of pavement structures, low-temperature cracks, and frost heaving. The traffic operation on this highway has been seriously affected by this pavement damage and distress. Based on 3 years of field observations and experimental tests of behavior, this paper emphasizes the proper design and construction of embankment and pavement with asphalt surface in permafrost regions of the Qinghai-Tibet Plateau. The minimum height of the pavement structure required for each pavement condition in permafrost areas is discussed. It involves study of the changeable patterns in the upper boundary of the permafrost table under different pavement structures, estimation of thaw settlement due to the construction of the asphalt pavement, and the seasonal change of structural capacity of the pavement. The minimum thickness of a pavement structure in permafrost regions of the Qinghai-Tibet Plateau is dictated by the type of pavement surface, condition of frozen soil above the permafrost table, annual air temperature, fill materials, and the criteria required for pavement performance. Pavement strength during a thaw can be determined by using the Benkelman beam deflection test. Tests show that the reduction in pavement strength during the thawing season is about 40 to 75 percent of the value during the freezing season. Finally, a practical evaluation procedure for pavement design and maintenance in permafrost regions of Qinghai-Tibet Plateau is presented.

Permafrost is present in approximately 2.14 million km^2 of China, which is over 22 percent of the total area of the country. In the extensive area of the Qinghai-Tibet Plateau, the so-called Roof of the World, there are large areas of continuous permafrost between the Himalayan and Kunlun mountains. The Qinghai-Tibet Plateau has the highest altitude, largest area, greatest thickness, and the lowest temperature among all the low-latitude continuous permafrost areas of the world. Its area is about 1.49 million km^2 and represents nearly 70 percent of the total area underlain by permafrost in China. Figure 1 shows the distribution of different types of permafrost and their areas in China. It also indicates the geographic location of the Qinghai-Tibet highway. Each type of permafrost is further classified into the following categories: continuous, discontinuous, isolated and alpine permafrost, as well as seasonally frozen ground (shown as the lightly shaded area).

Qinghai-Tibet highway, which is 860 km long, is one part of China's National Highway 209. It is a two-lane

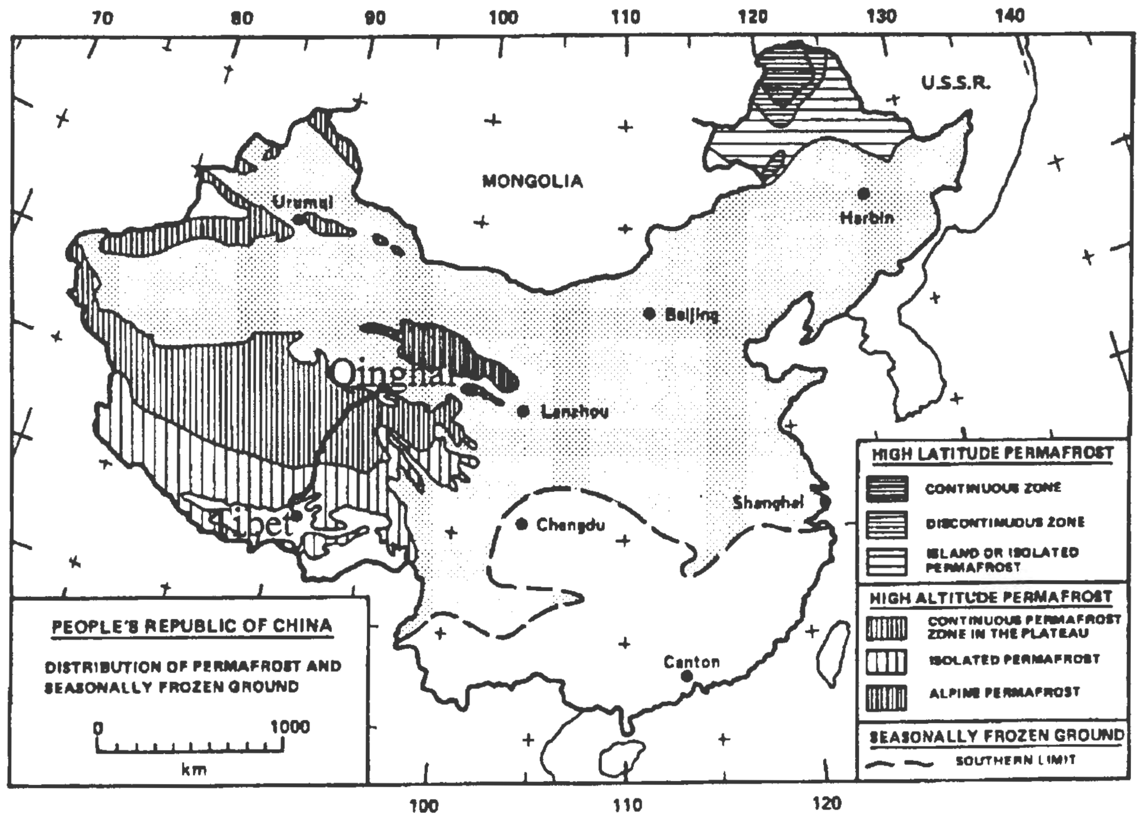


FIGURE 1 Outline map showing Qinghai-Tibet highway and distribution of permafrost type and area in China.

road with pavement widths of 9.5 to 12 m and is classified as a low-traffic-volume road with no more than 1,000 vehicles per day. The average elevation along the highway is approximately 4120 m. Certain sections have elevations of more than 5000 m, where the percentage of oxygen in the air needed for human activities is seriously deficient.

The large areas of perennially frozen ground at mid-latitude on the Qinghai-Tibet Plateau exist because the general altitude is 4000 m or more above sea level. Another major reason is that the frozen period of the ground surface in most regions of the plateau is as long as 7 to 8 months (*1,2*) Even in the warm season (from May to September), the ground surface freezes at night. For instance, in the area of the Kunlun Mountains, where the elevation is approximately 4700 m, during the warmest month of July, the highest temperature in the day is 17.6°C, while the lowest temperature at night is −8°C. Table 1 gives information on the climate in some of the permafrost regions of the Qinghai-Tibet Plateau, in which the permafrost table and thickness were investigated during the 1970s by Lanzhou Research Institute of Glaciology and Cryopedology of China (*3*) and the temperature was recorded by the Highway Bureau of Qinghai and the local meteorological bureau over a period of 20 years.

In the extensive permafrost areas of the Qinghai-Tibet Plateau, there exists a depth below the ground surface where the highest value of the ground temperature is zero. The ground above that depth thaws in the summer and freezes in the winter; hence, it is usually referred to as the "active layer." The ground below that depth is perennially frozen. The depth is then the thickness of the seasonally thawed layer, or the depth of the upper limit of the perennially frozen ground. The distance between the upper and lower limits of the perennially frozen ground is its thickness as shown in Table 1.

The highway was originally surveyed and built in the 1950s without detailed design to meet the urgent transportation need. There was very little research into the cost of permafrost engineering problems in the construction of the highway. In 1973 an asphalt surface layer was initiated; it took about 7 years to complete the resurfacing of the entire highway. However, some serious pavement damage or distress occurred immediately after the construction in the early 1980s. These problems included large amounts of thaw settlement of

TABLE 1 Principal Climate Factors in Some Regions of Qinghai-Tibet Plateau

Location of Permafrost	Elevation of Permafrost (m)	Thickness of Permafrost (m)	Depth of Active Layer (m)	Average annual air temperature (°C)	Average annual ground surface temperature (°C)
Tanggula Mt.	4950	164	1.2	-5.2	-2.1
Fenghuo Mt.	4700	120	1.1	-4.9	-3.7
Kunlun Mt.	4780	175	0.95	-5.7	-4.0
Qilian Mt.	4500	65	1.41	-5.5	-1.5
Bayan Shan	4430	78	1.61	-4.8	-2.7

embankments, unequal deformation of pavement surfaces, different types of cracks, and frost heaving, resulting in normal traffic operation being seriously affected. Consequently, a large-scale rehabilitation was required in the middle 1980s, which included raising the heights of embankments, clearing drainage systems, and resurfacing the pavement with asphalt concrete, which took about 3.5 years. Even with the rehabilitation, many permafrost engineering problems need to be resolved due to the original improper design and construction. These pavement maintenance problems from inadequate design and construction have been described (*4*).

To understand the factors that lead to the severe distress noted, a number of field experiments in four permafrost regions of the Qinghai-Tibet Plateau along the highway have been conducted since 1987 by the Highway Bureau of Qinghai. The purpose of these tests and observations was to determine the minimum height of a pavement structure under a specific condition in the permafrost regions of the plateau, including an analysis of factors that influence thaw settlement of the embankment, estimation of thaw settlement in quantitative terms, seasonal change of structural capacity during thawing and freezing cycles, and special evaluation of the pavement structure in the Qinghai-Tibet Plateau. Detailed discussions of these items is provided in the following sections.

Basic Considerations for Road Design and Construction

Climate Features

A preliminary investigation of geology and climate along the highway in the Qinghai-Tibet Plateau was conducted by the Highway Bureau of Qinghai and Lanzhou Institute of Glaciology and Cryopedology of China during the 1970s. The results of this investigation are shown in Tables 1 and 2. It has been found that most of the perennially frozen ground occurring along the Qinghai-Tibet highway is relatively stable (*2*). Therefore, any construction projects planned for this area should be based on the principle of preserving the permafrost, not increasing or even reducing the depth of the active layer. In addition, every effort should be made to leave the natural vegetation cover undisturbed (*5*). Except for the south area of the highway, the distinguishing features in these regions are (a) an annual evaporation three to five times greater than annual precipitation, meaning it is a very dry area; (b) a mean annual air temperature of less than -5.0°C; and (c) a high altitude with an average elevation of over 4500 m above sea level. Both the altitude and thickness of permafrost in the plateau obviously increase with a decrease in latitude (as the region approaches the highway mountains). Also at the same altitude, with each degree of decrease in latitude to the south, the average ground temperature was observed to increase by 0.7°C to 0.9°C.

There are essentially three types of soil in most permafrost regions of the Qinghai-Tibet Plateau: gravelly soil, sandy soil, and silty soil. Although the ground soil is basically sandy and gravelly in the Fenghuo Mountains and Tanggula region, clayey soil is extensively distributed in the Kunlun Mountains and Qilian region. The data obtained in these regions indicated that the gravelly soil in dry sections along the highway collects no water, consequently there is no serious frost heaving in the Fenghuo Mountain region, but thaw settlement is still a major problem in all regions of the plateau. Clayey soil, on the other hand, accumulates additional

TABLE 2 Thickness of Permafrost in Some Regions of Qinghai-Tibet Plateau

Location or Region	Latitude	Mean Annual Air Temp.(°C)	Annual Precipitation (mm)	Annual Evaporation (mm)	Period of Test
Liandao He	31°52'	-1.1	892.5	1626.2	1986-89
Tanggula Shan	32°51'	-5.4	476.1	1897.2	1975-82
Fenghuo Shan	32°45'	-5.1	324.7	1427.9	1978-82
Kunlun Shan	35°29'	-6.2	434.7	1600.1	1982-87
Qilian Shan	38°17'	-5.3	644.5	1626.8	1984-89
Jiang Cang	38°21'	-3.1	619	1624.2	1984-89

water when it is in a frozen state, hence frost heaving will always occur in the cold season due to volume expansion.

Distribution of Temperature Under Soil

Since asphalt pavement absorbs more solar radiation than gravel road surfaces, the ground soil under a newly paved asphalt highway will thaw earlier and freeze later than it had before the asphalt surfacing. The upper boundary of the permafrost will decline downward by depths ranging from about 120 to 750 mm, depending upon the thickness of asphalt surface, the number of years it has been paved, the annual air temperature above the ground soil, the height and orientation of embankment, and the embankment materials. The increased depth of the active layer will, in turn, result in a greater thaw settlement of the pavement structure.

Figure 2 shows the seasonal changes in thawing and freezing depths of the ground soil in the Qinghai-Tibet Plateau. The field tests were done in the Fenghuo Mountain region on two different pavement sections. One section was paved with a 40-mm thickness of asphalt, while the other section was a gravel surface. Both had the same total thickness of 520 mm. The test sections were instrumented with thermocouples and resistivity gauges to determine the location of the 0°C isotherm and the location of the freezing or thawing front, respectively. Data were collected during the 3-year period from 1988 to 1991. The results indicate that there is a general pattern of temperature distribution in ground soil under different pavement conditions in the permafrost area of the plateau. It should be noted that the thawed depth varies with the thickness of pavement structure, especially the thickness of the asphalt concrete surface; annual temperature; climate; and type of subgrade soil under the surface. The following facts can be deduced from Figure 2.

1. Ground soil with an asphalt surface could result in a maximum of about 1 month of thawing with a delay of 20 to 30 days for freezing, depending on the thickness of the asphalt concrete surface, the annual aboveground temperature, and the age of the asphalt surface. Generally, in the initial 2 years, this phenomenon is more marked; in subsequent years, the difference becomes gradually smaller. When a new asphalt surface is applied as a routine maintenance, this phenomenon again emerges.

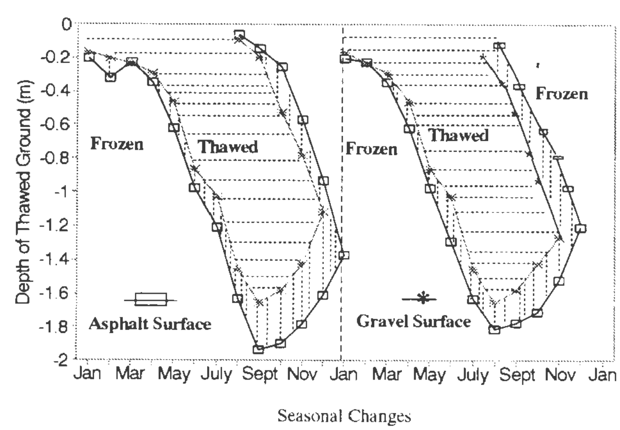


FIGURE 2 Seasonal changes in freezing and thawing depth of frozen ground soils underlying asphalt surface and gravel surface in Fenghuo Mountain region.

2. The maximum thaw depth caused by the asphalt surface is about 200 to 700 mm deeper than that of the gravel surface, depending on the thickness of asphalt surface, the width of pavement, annual air temperature, orientation of the embankment, and content of water or ice in the newly thawed soils. The maximum thawing depth of the ground with asphalt surface will decrease with age, while the maximum thawing depth of the ground with gravel surface will not significantly change with age provided the mean annual air temperature remains approximately the same each year.

3. The frozen time can also be delayed 15 to 30 days by the asphalt surface. The completion of the freezing is normally in mid-December under the gravel surface, while it can be shifted to mid-January under the asphalt surface.

4. In late August or early September of each year, the ground soil starts to freeze from both the upper side, where the air temperature is below freezing, and the lower side, where it is close to the frozen upper boundary of the permafrost table. The entire ground soil freezing process will be completed in December or January of the following year.

In addition, the relationship between thaw depth and ground temperature for three different pavement conditions has been observed in the same permafrost area. The tests were performed in May and August, in which the average air temperature is the lowest and highest, respectively. The test results are shown in Figure 3. It is obvious that the thaw depth increases when an embankment is constructed. The tests also indicate that thaw depth under an embankment with an asphalt surface will increase much more than that of an embankment with gravel, although the total heights of the embankments are approximately equal. In this field experiment, the total increased depth by the embankment with asphalt surface is about 850 mm in the first year and 700 mm in the second year, compared with the natural ground (no embankment).

Thaw Settlement and Shift of Permafrost Table

The main problem in this area was to maintain the stability of the roadbed through the permafrost zone. Over 200 km of the highway crosses an area containing significant amounts of ground ice. A key question was how to keep the paved highway from being damaged when the ground ice melts. Other important issues included protecting the highway from frost heaves and frost boils and accurately determining the optimum embankment elevation because over 90 percent of the damage occurring to the old Qinghai-Tibet road was caused by insufficient embankment elevation and poor drainage.

Generally thaw settlement is the differential downward movement of the ground surface resulting from the melting of excess ice in the soil and the thaw consolidation of the soil mass under its own weight and external loadings. A one-dimensional finite element model was developed to estimate thaw depths and subsequent settlement of an embankment in a permafrost terrain (6). In permafrost regions, the main cause of damage to highways is thaw settlement, which can be classified as sudden or gradual in terms of duration and process.

Sudden Thaw Settlement

Sudden thaw settlement occurs when roads are built in a region where the upper portion of the permafrost gen-

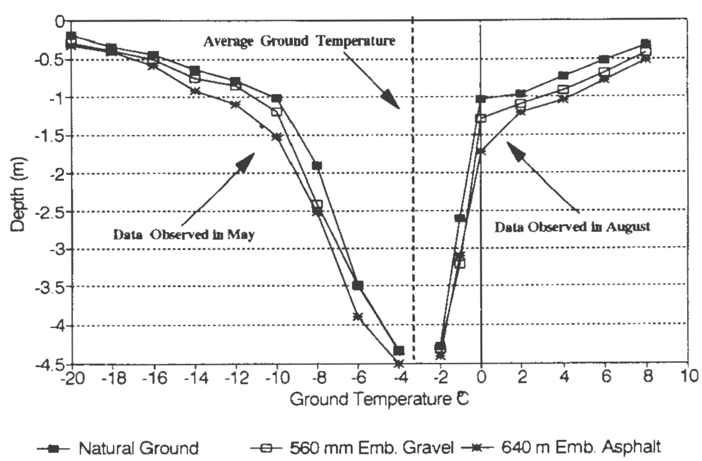


FIGURE 3 Comparison of temperature in ground soils under different heights of embankment.

erally contains a thick clayey, ice-rich soil. When the ice melts, the soil reaches a supersaturated state, and the pavement structure may lose almost its entire bearing capacity. The result is a large amount of subsidence under external loads and its own weight. In addition, when the depth of thaw is unequal on both sides of the embankment, a slanting, frozen slip plane is formed beneath the subgrade. Under these conditions, the vibrations and loads of passing traffic may force the supersaturated clayey soil to be squeezed out along the frozen slip plane so that a sudden slump will occur in an embankment that had been gradually subsiding during a warm period. Sudden settlement can cause drivers to lose control of their vehicles.

Gradual Thaw Settlement

Gradual thaw settlement follows sudden thaw settlement. Gradual settlement is dictated by the density of the frozen soil, height of the embankment, fill materials, loading, and traffic. In addition, when water infiltrates the road embankment and accumulates on top of the permafrost table during the rainy season, ground ice in this area will thaw slowly but steadily; this, in turn, can cause the pavement to gradually subside. Subsidence of a pavement surface from this type of thaw settlement makes road maintenance work very difficult and costly. For example, thaw settlement of an embankment of 1.15 m in height has been observed in the Kunlun Mountain region. Records of accumulated thaw settlement showed that the process of gradual thaw settlement of an embankment has continued for 12 years since the construction of the road, although at a declining rate.

A general movement of thaw depth or shift of the permafrost table after construction of the embankment and then the asphalt surface layer is illustrated in Figure 4. When the embankments constructed above the natural ground are greater than a minimum (critical height) thickness, the permafrost table will rise, by certain amounts, depending on the height of embankment. Figure 4 is representative only when the height of the embankment is greater than a minimum height required in a specific permafrost region. If the height of an embankment is less than the minimum value, the level of Boundary Line 3 (upper boundary after paving with asphalt surface) could be below Boundary Line 1 (depth of permafrost table under natural conditions), which means the upper portion of the permafrost table will melt in the warm season. Some field observations of change in permafrost table were conducted in the Fenghuo Mountain region for a period of 3 years. Table 3 contains the observed data. The value of the raised upper boundary after construction of the embankment is directly propotional to the height of the embankment (see Figure 4), and the amount of the reduced thaw depth is the difference between Boundary Line 2, which is raised due to construction of the embankment, and Boundary Line 3, which has moved down because of paving with an asphalt surface.

Since the prediction of thaw settlement is considered a critical element in pavement structure design and the thickness calculation of each layer, some empirical equations for estimating the thaw settlement of a pavement structure in each specific permafrost region of the Qinghai-Tibet Plateau have been developed by means of regression analysis. It is beyond the scope of this paper to describe the detailed procedures and equations of the regression analysis. Figure 5 shows the observed thaw settlements for a period of 3 years on three different thickness of pavement structure in the Fenghuo Mountain region. It illustrates that the thaw settlement could be controlled if the height of embankment is adequate and the quality of fill materials is good.

Frost Heaving of Pavement

Frost action is a complex physical and chemical process. The physical component is the dominant one and includes the transmittal of heat, the phase change of water, water migration, and various mechanical processes under certain conditions of loading, temperature, and moisture.

The mechanics of the frost heaving phenomenon include (a) frost-susceptible soil, (b) slowly depressed air temperature, and (c) water supply. The amount of frost heave is defined as the increment in the vertical direction. Increase of the ice volume is a necessary condition for producing frost heave (7). If water from an external source is supplied to the subgrade, there will be a conspicuous increase in the water content of all the soil comprising the subgrade and subbase. When water contained in the subgrade soil freezes, it expands in volume. The expansive force generated by the freezing of the subgrade soil will push the pavement surface upward

According to observations in some permafrost regions of the Qinghai-Tibet Plateau, frost heaving of the highway subgrade usually occurs when the embankment is low and built on grassy, water-logged marshland. These types of soils are generally sandy or clayey and frequently have a high moisture content. When this moisture freezes, pavements are very easily elevated to different heights, disrupting highway traffic. Consequently, the problem of frost heave in permafrost areas can be reduced to a great extent by preventing external water from entering the pavement structure.

Table 4 contains data for the maximum frost heave of the pavement surface under different conditions along the Qinghai-Tibet highway in the Tanggula and

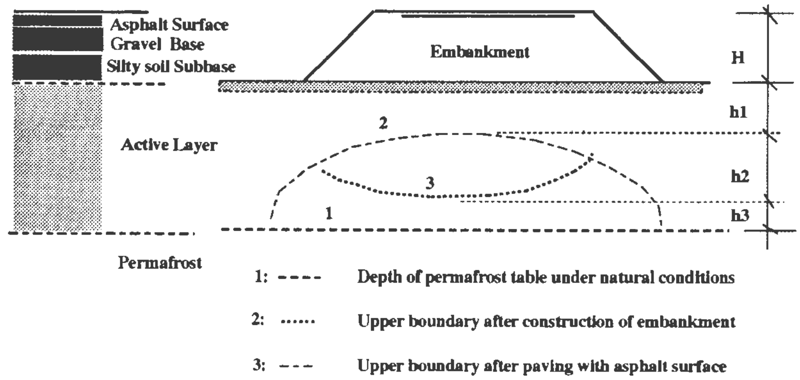


FIGURE 4 General movement of permafrost table after highway construction in Fenghuo Mountain region of plateau.

TABLE 3 Maximum Frost Heave Observed in Roads Under Different Conditions of Frozen Soil and Water Supply

Observed Experimental Section No.	Depth Under Natural Condition H_1 (cm)	Height of Embankment (cm)	Depth Predicted After Road Construction (Estimated by Empirical Formulas), H_2 (cm)	Depth Observed After Paving Asphalt Surface, H_3 (cm)	Decline of Upper Boundary of Permafrost (cm)
T100+1F.Shan	2.23	84	56	22	32
T100+2F.Shan	2.56	75	42	19	23
T100+3F.Shan	1.86	115	78	57	21
T100+4F.Shan	1.34	120	84	62	22
T100+5F.Shan	3.10	64	32	-7	39
T100+6F.Shan	3.58	52	28	-10	38

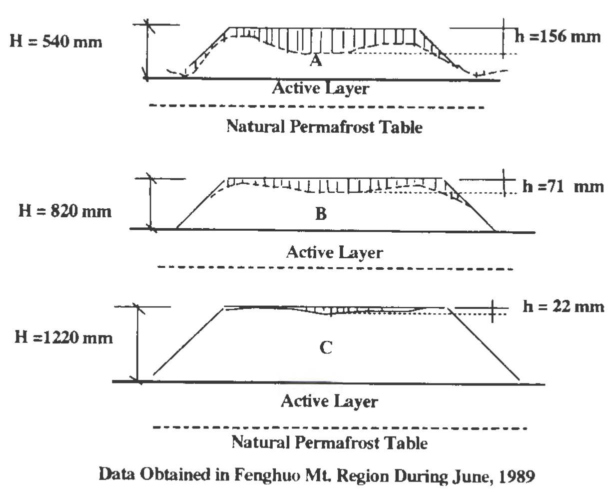


FIGURE 5 Deflection measurements by Benkelman beam through a year in Fenghuo Mountain region of the Qinghai-Tibet highway.

Fenghuo Mountain regions. The amount of ground heaving is clearly related to the level of the water accumulating beside the road or to the level of the groundwater table before freezing. Both of the two largest frost heaves obtained from the observation took place in the areas where the extra water can be supplied from the pool along the roadside.

Pavement Evaluation in Permafrost Areas

Deflection Measurements

Benkelman beam deflection measurements were conducted in the Fenghuo Mountain region for a period of 3 years (1989–1992). As shown in Figure 6, the result of the deflection measurement indicated that the reduction in bearing capacity was strongly dependent on the

TABLE 4 Change of Upper Boundary of Permafrost in Fenghuo Mountain after Asphalt Road Construction

Observed Regions of Road section	Pavement Structure and Thickness of Each Layer	Condition of Frozen soil above Permafrost Table	Height of Road Surface Above the Level of Pool Water Along the roadside (m)	Difference Between Road Surface and Ground Water Before Freeze-up (m)	Maximum Frost heave observed within 3 years (mm)
Test #1 (Fenghuo Mt.)	38 mm asphalt 220 mm gravel base 280 mm sandy soil	Ice-rich frozen soil	0.35	0.86	131
Test #2 (Fenghuo Mt.)	47 mm asphalt 300 mm stabilized gravel 380 mm sandy soil	Ice-saturated frozen soil	0.53	0.92	74
Test 3# (Fenghuo Mt.)	47 mm asphalt 350 mm binder and gravel 450 mm sandy soil	Ice-saturated frozen soil	1.25	1.04	42
Test #1 (Tanggula Mt.)	52 mm asphalt 150 mm gravel 35 mm old asphalt 350 mm binder and gravel 450 mm clayey soil	Ice layer with soil inclusions	0.65	1.10	55
Test #2 (Tanggula Mt.)	40 mm asphalt 250 mm gravel 35 mm old asphalt 300 mm stabilized soil 380 mm clayey soil	Silty soil with vegetation cover removed Ice layer with soil inclusions	0.85	0.75	97
Test #3 (Tanggula Mt.)	40 mm asphalt 300 mm binder and gravel 450 mm clayey soil	Ice-saturated frozen soil	0.46	0.87	145

thickness of pavement structure if other conditions were equal. The lowest strength of pavement structure took place in June or July when full thaw had occurred. For example, the pavement section on clay subgrade (55 cm in height) had a maximum deflection in June almost four times larger than that measured in January. For the other two conditions, the values of the maximum deflection are two to three times greater than that tested in January or February, when the pavement strength is highest. Additional work is required to determine the correlation between the freezing index and thaw depth.

Minimum Height of Pavement Structure

The minimum height of pavement structure in a permafrost area should be sufficient to prevent the natural permafrost table from declining. If ice-rich frozen soil is considered the basic condition of the ground soil, a certain amount of height should be added or reduced if the frozen soil is classified as having a higher or lower water content. For example, the minimum thickness of pavement structure constructed on three different frozen soils in the Fenghuo Mountain region of the Qinghai-Tibet highway is shown in Figure 7. Observations from experimental sections in the Fenghuo Mountains, Wudaoliang, and Tanggula Mountains showed that the upper limit of the permafrost underlying the newly paved asphalt roads fell to a level that was 450 to 750 mm lower than the upper limit of permafrost lying beneath nearby roads with a gravel surface.

"More filling and less excavating" is the principle derived from extensive design and construction experience in the permafrost area of Qinghai-Tibet Plateau. When the climate is sufficiently cold, the thaw period is short, and hence the depth of annual thaw is shallow. In the permafrost area of the Qinghai-Tibet Plateau, the annual average air temperature is −6.7°C; therefore, complete protection against the detrimental effects of frost action—especially of permafrost degradation—can be achieved by using nonfrost-susceptible materials for fill to a proper height.

The natural upper boundary of the underlying permafrost may change after road construction because the thermal equilibrium of the original frozen ground might be disturbed. To ensure that the upper boundary of the permafrost does not change after an asphalt surface has been applied, the road embankment should be elevated to a height 0.7 m higher than that of the old road.

Since ground ice is a major component of frozen soils in the plateau area, it has tremendous influence on the

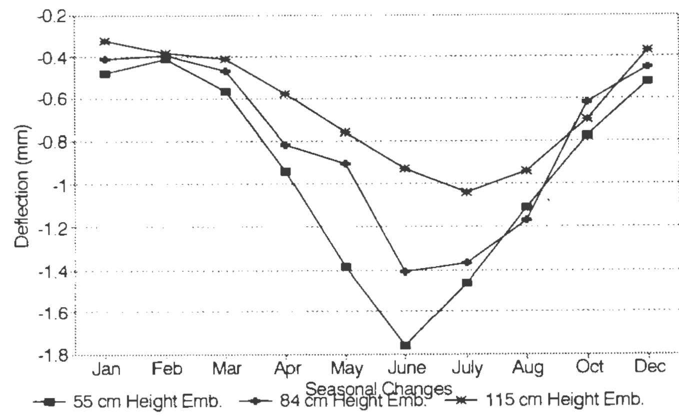


FIGURE 6 Measured thaw settlements of pavement structure under different conditions in south permafrost areas of Qinghai-Tibet Plateau.

stability of the pavement structural capacity during seasonal thawing and freezing. The water migration and the different types of segregation ice during the freezing period were observed in 35 boreholes and 27 test pits. In these regions, three types of segregation ice in terms of structure were found through a microscopic analysis: granular segregation ice, cementation ice, and laminar segregation ice. The ice content of segregation varies with different types of ground soils and geographic location.

Experiments conducted in the Fenghuo and Tanggula Mountains indicate that thaw settlement resulting from an asphalt pavement surface may be prevented by raising the embankment. However, other problems will appear if the embankment is too high, such as compaction, stability, safety, construction, and so on. In other words, both the minimum height and maximum height of an embankment in a permafrost area must be considered.

Proposed Model for Pavement Evaluation in Permafrost

A performance evaluation of a pavement structure in the permafrost areas of the Qinghai-Tibet highway includes not only criteria on riding comfort, structural capacity, skid resistance, and so on but also thaw settle-

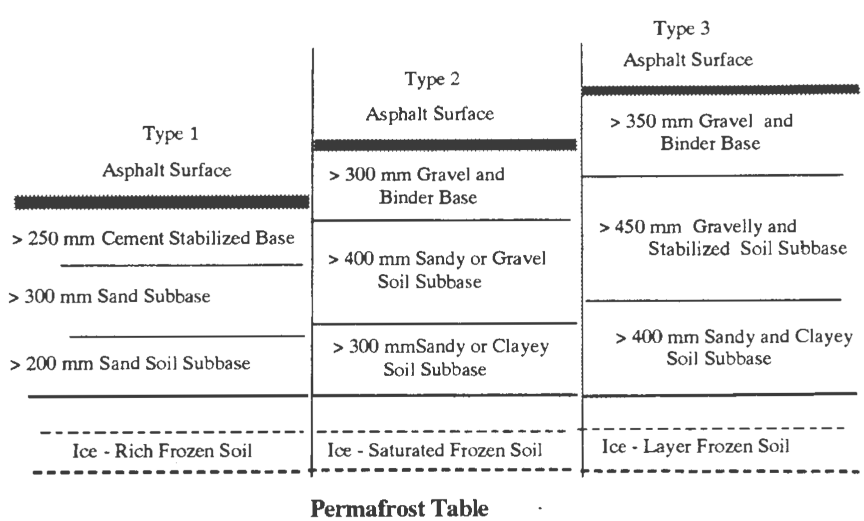


FIGURE 7 Recommended height of embankment under three different ground soils in permafrost regions of Qinghai-Tibet Plateau.

ment and frost heave criteria. Like roads built in many other permafrost areas of the world, pavement roughness caused by the uneven settlements due to thawing is the major problem in the Qinghai-Tibet Plateau. The highway design and construction in continuous permafrost areas should be predicated on the principle of soil in its frozen state needing to be preserved.

Figure 8 is a schematic diagram of the proposed model for evaluating pavement structure in the permafrost areas of the Qinghai-Tibet Plateau. The key components and considerations for pavement design are described by three sets of criteria: permafrost criteria, distress criteria, and performance criteria. Under each set, there are four levels of assessment on the structure design. For example, under the permafrost block criteria, all factors related to permafrost conditions (permafrost table, thickness of pavement, type of frozen soil, and water supply) are evaluated. Condition of water (either in the form of ice crystals or free water state) in the soil may be determined by the temperature of the permafrost soil; the bearing capacity of the soil can be decided by the amount of ice crystals in the permafrost soils.

For the purpose of assessing the engineering characteristics of frozen soils, a geotechnical classification of permafrost in the plateau and some major considerations in pavement design should be carried out. A simple classification was based mainly on many natural soil laboratory experiments and a few field experiments. A statistical relationship between thaw settlement and groundwater content was evidenced in the thawing and freezing processes in frozen soil. It is helpful to analyze the principal factors in highway construction failures, such as thaw settlement, frost heave, and seepage of water. Some other considerations and methods associated with the design and construction of pavement structures in the Qinghai-Tibet Plateau are summarized in the following paragraphs.

When a pavement initially built of gravel materials is paved with asphalt, much more heat is absorbed from sunlight through the asphalt layer. Therefore, the upper boundary of the permafrost table will decline, and new thaw settlement has to be considered again. Even if the old pavement has already been surfaced with asphalt, any maintenance operation (i.e., a resurfacing treatment with asphalt) will shift the upper boundary of the permafrost table down to a certain magnitude of depth.

Orientation of highway geometric alignment design is a very important factor affecting thawed depth of a permafrost embankment. For instance, two sections of highway with different orientations (one is east-west; the other one is south-north) in the Fenghuo Mountains were observed; the difference of thaw settlement between the two sides of embankment was found to be 200 to 500 mm.

Elevating the grade line sufficiently to ensure that the sides of the roads are above the groundwater table—or above the level of the water accumulating beside the road—is important. To keep frost boils and frost heaves from occurring as a result of the infiltration of

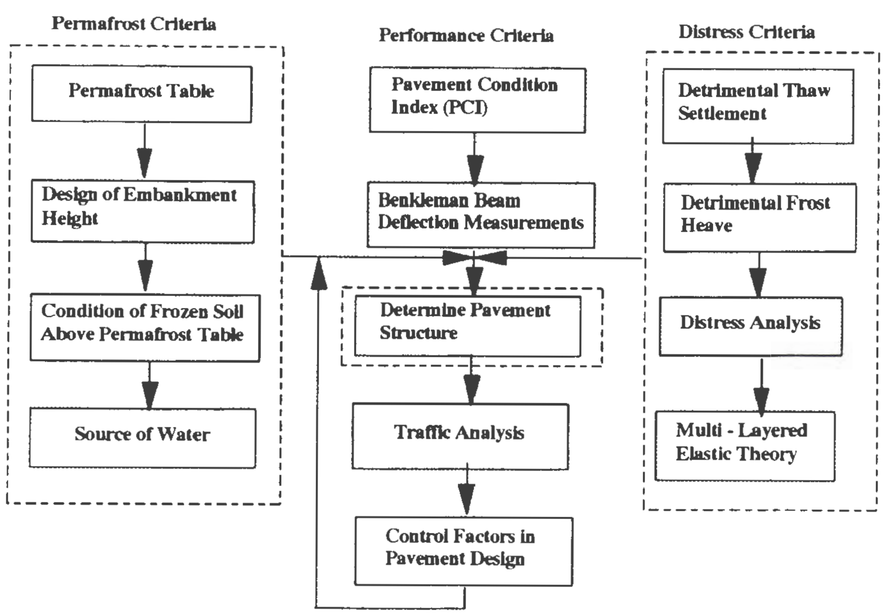


FIGURE 8 Proposed pavement evaluation procedure in permafrost regions of Qinghai-Tibet Plateau.

surface water, the asphalt pavement must be impermeable to water, and the shoulder of the road should be strengthened to ensure good drainage.

Adjusting the properties and thicknesses of each layer of the pavement structure to ensure the predicted stresses, strains, and deflections are within allowable limits is another major consideration. The mechanistic approaches provide flexibility in modeling a material, and drainage conditions in the field may be incorporated in the analysis.

The formation of cracks in the pavement surface not only depends upon the air temperature but is also influenced by thaw settlement of the subgrade. Hence, the amount of settlement of the pavement surface depends on the distribution of ice in the frozen soil and the swelling characteristics of the skeleton of the thawed soil. The amount and distribution of ice and the stress-deformation behavior of the thawed soil depend upon a variety of factors, including the stress, thermal, and moisture change histories of the material.

Conclusions

In the extensive permafrost areas of the Qinghai-Tibet Plateau, pavement maintenance or rehabilitation is greatly affected by the original design and construction of the highways. It is very important to consider all major factors that influence the basic performance of the pavement structure before starting road construction on ground with permafrost. Otherwise, some severe damage or distress of the pavement structure may result immediately after construction; in some cases, maintenance may be impossible because the original design and construction were improper. The 20-year experience of pavement maintenance along the Qinghai-Tibet highway has generated the following information.

1. Thaw settlement of a highway subgrade is directly related to the construction of the embankment and the type of frozen ground soil (ice-rich frozen soil, ice-saturated soil, or ice layer with soil inclusions) above the permafrost table. Therefore, under each type and condition of frozen ground soil, the amount of thawing settlement of the pavement structure can be determined by the height of embankment, the fill materials of each layer, the orientation of the center line of the embankment, and the annual air temperature.

2. There exists a minimum height of embankment that could prevent the upper boundary of permafrost from shifting upward or downward for each situation in a permafrost area. For instance, in Fenghuo Mountain permafrost regions of the Qinghai-Tibet Plateau, the minimum height of an embankment is estimated to be 0.7 m when the pavement structure is a 42-mm asphalt surface; the subbase and base filling material is composed of sandy or gravelly soil, respectively; and the orientation is 15°24′ northeast. If the height of the embankment is less than the critical height, the depth of the permafrost table will decline and result in thaw settlement.

3. When asphalt concrete or even asphalt seal is to be used to resurface the existing pavement (even if the old surface is asphalt) during routine maintenance, the existing permafrost table will shift downward because asphalt can absorb and conduct much more heat from sunlight.

4. Frost heave along the Qinghai-Tibet highway is mainly the result of water seeping in through the cracks in the pavement surface and being stored in the upper layers during the warm season and freezing with volume expansion in the cold season. This distress can be minimized by sealing cracks in the pavement surface and providing a good drainage system.

5. Defection measurements indicate that the bearing capacity of the pavement structure in the warm season (June and July) in some permafrost regions of the plateau will lose 20 to 70 percent of its value compared to the cold season.

6. The behavior of asphalt surface pavements in permafrost areas can be predicted by evaluating the three sets of criteria for pavement structural design: thaw settlement criteria, frost heave criteria, and performance criteria. The proposed pavement evaluation procedure for the Qinghai-Tibet highway in the permafrost regions is practical. The performance evaluation in permafrost areas should include, in addition to the common requirements, a cumulative damage procedure due to thawing and freezing cycles.

ACKNOWLEDGMENTS

The authors would like to thank the Highway Bureau of Qinghai for all its support during the data collection and technical research on road maintenance in permafrost regions of the Qinghai-Tibet Plateau. Appreciation is also extended to the Natural Sciences and Engineering Research Council of Canada for providing financial assistance to Li Ningyuan.

REFERENCES

1. Wu, Z. Discussions and Opinions on the Article: A Geotechnical Classification of Permafrost. *Journal of Glaciology and Cryopedology*, Vol. 2, No. 2, 1980, pp. 58–60.
2. Wang, J., W. Xiaoling, and Q. Guoching. The Multi-Year Permafrost Along the Qinghai-Xizang Highway. *Geography Journal*, November 1978.

3. Cheng, G. Certain Distributions Between the Permafrost of the Chinese Qinghai-Tibet Plateau and That of the Canadian North. *Journal of Glaciology and Cryopedology*, Vol. 2, 1979, pp. 39–42.
4. Li, N., and Bin Li. *Feasibility Research of Design and Construction on Highway Tunnel in the Permafrost Region of Qilian Mt.* Technical Report. Highway Bureau of Qinghai. 1990.
5. Johnston, G. H. *Permafrost Engineering Design and Construction.* Wiley, 1981.
6. Hildebrand, E., and R. Haas. Thaw Settlement and Ground Temperature Model for Highway Design in Permafrost Areas. *Proc., Roads and Transportation Association of Canada*, Edmonton, Alberta, 1983.
7. Berg, R. L. *Optimized Freeze-Thaw Design of AALRS.* Air Force Engineering and Tyndal Air Force Base, Fla., 1988.

PAVEMENTS

Overview of Minnesota Road Pavement Structure Research Project

David M. Johnson, *Minnesota Department of Transportation*

The Minnesota Road (Mn/ROAD) research project test facility by design and location supports cold regions pavement research for both low- and high-volume roads. The facility is one of a kind and is impressive in terms of size and automation. However, the real payoff for the $25 million investment lies in the results from research conducted there. The facility was completed and opened to traffic in June 1994. Research activities are under way. Extensive materials testing and baseline readings of sensors have been completed, yielding preliminary results as well as additional questions. Unique and leading-edge engineering tools used or developed on this research project are being implemented for the operational needs of Minnesota pavement and materials engineers. For more long-term results, a pavement structure research program for Mn/ROAD is in place. Besides performing research with inhouse staff, the Minnesota Department of Transportation has substantial cooperative research agreements with the University of Minnesota, FHWA, the U.S. Army Engineer Cold Regions Research and Engineering Laboratory, and the Finnish National Road Administration. The general objectives of Mn/ROAD research are to evaluate electronic sensors and other pavement characterization tools, verify existing pavement design and analysis models, investigate factors that affect pavement response and performance, and develop new and improved pavement design and analysis models.

The purpose of this technical note is to give an overview of the background, direction, and content of the pavement structure research program at the Minnesota Road (Mn/ROAD) research project facility. Mn/ROAD is a full-scale outdoor pavement research laboratory operated by the Minnesota Department of Transportation (Mn/DOT) in the city of Otsego, Minnesota.

The facility is designed to provide pavement researchers with data on traffic loadings, environmental conditions, construction material characteristics, pavement response, and pavement performance throughout the next 20 years. The purpose of the Mn/ROAD research program is to coordinate and organize research activities and results from a multitude of active and proposed projects to achieve the goal of Mn/ROAD through a series of appropriate and implementable steps. The goal of Mn/ROAD is to make the design, construction, and maintenance of pavements more effective and efficient in cold regions through the development of improved pavement analysis and design procedures.

Background

The last major full-scale road laboratory in the United States was the AASHO road test facility constructed in Illinois in the late 1950s. The results from the AASHO experiment form the basis for most pavement structure designs today. Since that experiment there have been a number of significant changes:

1. There is a greater awareness of the effects of weather and climate on the performance of pavement structures.
2. There is a greater awareness of the effects of variability on the reliability of pavement models.
3. Electronic instrumentation has emerged as a reliable method for monitoring the responses of pavement structures.
4. There is now a strong interest in developing reliable, mechanistic-based pavement models.
5. The characteristics and economics of available pavement materials have changed.
6. Pavement construction methods and equipment have changed.
7. Traffic demands have increased and load characteristics have changed.
8. The Strategic Highway Research Program (SHRP) Long-Term Pavement Performance (LTPP) program was initiated and continues under FHWA's direction
9. An era dominated by new highway construction has ended and an era of highway rehabilitation and reconstruction has begun.

Furthermore, in describing the role of SHRP, SHRP Executive Director Damian Kulash pointed out that despite the fact that over 20,000 pavement research papers were catalogued during the past 10 years by TRB, there were no major improvements in pavement life or durability during that period (*1*). This last statement is debatable, but it does highlight a desire for breakthrough changes in pavement engineering. All the above points dictate that a new pavement structure experiment is needed.

Direction

Frank McCullough of the University of Texas at Austin describes an approach for developing and verifying pavement models (*2*). It calls for increasing levels of model verification in terms of cost and confidence, ranging from laboratory materials characterization tests to accelerated pavement structure testing and finally to long-term, full-scale road experiments. Results at each level are fed back into the model to refine it. Mn/ROAD offers key components in this approach to developing reliable mechanistic-based pavement design procedures for cold regions.

Since the design and construction of the Mn/ROAD facility were initiated and accomplished primarily with Mn/DOT resources with strong support from the University of Minnesota (UM) and the Minnesota Local Road Research Board, the vision for Mn/ROAD research appropriately focuses on the benefits for the citizens of Minnesota. This vision is as follows:

> . . . to develop the expertise and the tools that will confront and solve the pavement structure engineering and economic problems of Minnesota in the 21st century. (*3*)

However, it has never been the intention of Mn/DOT to restrict the benefits of Mn/ROAD to Minnesota. On the contrary, Mn/DOT perceives the success of Mn/ROAD research as directly proportional to the amount of support it offers to other transportation agencies, educational institutions, and organizations interested in pavements. Of particular importance are nearby states and other areas with conditions similar to those in Minnesota.

FHWA has provided substantial financial support for Mn/ROAD construction and research. Other federal agencies of the United States and foreign countries support the project. Several state transportation departments and universities have supported work or visited Mn/ROAD. As the project has developed, more people and organizations have become involved. The design of the facility reflects the input of over 100 individuals serving on 10 committees.

The Mn/ROAD facility includes 40 test sections, each about 150 m (500 ft) long. Each test section is made up of a different combination of material layers. Surface layers include asphalt concrete (22 sections), portland cement concrete (14 sections), aggregate (2 sections), and chip-sealed (2 sections).

The Mn/ROAD test sections do not constitute a full factorial experiment. Rather, test sections represent diverse cells in the large factor matrix associated with Mn/ROAD. For example, asphalt concrete section design factors are as follows:

- Seven pavement thicknesses (76 to 273 mm);
- Two subgrade soils (R = 12, 70);
- Fifteen subbases (various materials and thicknesses);
- Two drain states (drained, undrained);
- Two pavement viscosity numbers (PVNs)—low, high; and
- Four lab compactions (low, medium, high, very high).

Portland cement concrete section design factors are as follows:

- Four pavement thicknesses (152 to 241 mm);
- Two subgrade soils (R = 12, 80);

- Five base type combinations;
- Two drain states (drained, undrained);
- Four dowel diameters (0 to 38 mm);
- Four panel lengths (3.7 to 7.6 m);
- Three pavement widths (7.6 to 12.2 m); and
- Two supplemental steel states (added, not added).

There are actually 80 test sections at Mn/ROAD, since each test area has two lanes that will experience different loadings. An extensive testing and instrumentation program at Mn/ROAD further defines the spatial and temporal variability of project characteristics.

During construction approximately 4,700 material samples were collected for research purposes. Mn/DOT, UM, and Braun Intertec laboratories have completed thousands of routine material characterization tests, from gradation to M_r. The remaining samples are stored for future testing.

Mn/ROAD staff installed 4,572 electronic sensors, including 22 different sensor types, throughout the pavement structure to measure static and dynamic responses to environmental and traffic conditions. Some sensors are monitored automatically on a regular basis and others are monitored intermittently as needed for specific research work. The following measurements are made throughout the pavement structures:

- Static and dynamic strains,
- Static and dynamic pressures,
- Surface and subsurface deflections,
- Vertical accelerations at joints,
- Slab tilts,
- Temperature profiles,
- Total moisture content profiles,
- Unfrozen moisture content profiles,
- Soil suction profiles,
- Frost front locations,
- Water table locations, and
- Subsurface drainage outflows.

An automated weather station at the site regularly collects various weather data, which will be collected with nondestructive equipment during the life of the pavement structures:

- Falling-weight deflections,
- Roughness profiles,
- Cross-section profiles,
- Distress surveys,
- Visual images, and
- Skid resistance.

Approximately 40 gigabytes of data will be collected each year. A relational data base structure and graphical user interface support the researchers performing the analyses.

Twenty-three of the 40 test sections are loaded with westbound traffic on Interstate Highway 94; 14 of these 23 Interstate sections are expected to have about a 10-year life, and the remaining sections should last 5 years. An automated weigh-in-motion scale monitors these loadings. An average of 7,000 heavy commercial axles are westbound on a daily basis. Seventeen of the 40 test sections are in a closed loop and are loaded with calibrated trucks. One lane will be loaded 4 days a week with a maximum legally loaded truck at 356 kN (80,000 lb). The other is loaded 1 day a week at 498 kN (112,000 lb). Equivalent single-axle load (ESAL) accumulation rates are approximately equal for the two lanes. These thinner, low-volume sections are expected to last about 3 years.

In light of the importance of research on pavement rehabilitation and reconstruction, committee work began in this area over a year before the Mn/ROAD facility was opened to traffic.

Content

The content and quality of the Mn/ROAD research program are dictated to a large degree by the capabilities of the facility and the quality of the generated data. Therefore the first research objective was to build the project well. For this reason researchers were heavily involved in the design and construction of the facility, particularly in the area of electronic sensors and materials testing. Such intimate knowledge of the facility from a research perspective is of great value to all future research work.

The variety and quantity of data generated by the Mn/ROAD facility can be used for a vast number of research projects, many of which may not have been conceived yet. Unlike the SHRP LTPP test sections, the Mn/ROAD facility is geared to provide pavement researchers with continuous or nearly continuous dynamic and static pavement response data, as well as performance data. It allows the detailed analysis of daily or even hourly pavement structure events.

Of course, the primary objectives of the Mn/ROAD research program relate to conducting research projects that contribute to the realization of the vision for Mn/ROAD research. In order to bring some structure to the program, projects are classified as those that verify existing pavement models, those that contribute to understanding factors affecting pavement response and performance, and those that develop design procedures with improved mechanistic and statistical functionality. What follows is a description of some of the significant components of the Mn/ROAD research program in each of these classifications.

An important component of any research project evaluating a new method is a comparison with the current method, or control method. Similarly, the Mn/ROAD research program includes studies that evaluate current models for designing pavement structures. Although current design tools are beginning to include some aspects of mechanistic design, they are primarily empirical in nature and base their predictions of pavement life on the results of the 40-year old AASHO road test. Of primary interest in this area are the evaluation of the Minnesota pavement design method and the AASHTO Guide for Design of Pavement Structures. Two recent TRB papers highlight significant concerns with the flexible pavement design methods of the AASHTO guide (*4,5*).

Another key component of any research project is simply learning through analysis. What is learned is then correlated and consolidated in the research project findings. On a parallel track, many of the Mn/ROAD research projects are designed to uncover specific types of information that are woven together and fed into other projects. Some of these "building-block" projects are short-term projects that address pavement response questions and long-term projects that address pavement performance (life) questions, whereas some attempt to develop relationships between the response and performance. Some projects develop mechanistic material parameters indirectly through field testing and backcalculation, whereas others measure these parameters directly with laboratory tests. Still others describe and predict the state of various pavement structure components under a variety of conditions. Two major (and a few minor) building-block projects address seasonal changes and various truck loading configurations

NCHRP Project 1-26, developed primarily at the University of Illinois, describes in general terms the types of models, relationships, and input needed for the development and calibration of a mechanistic pavement design procedure (*6*). This report also highlights the need for a closed-loop approach to validate the procedure. The NCHRP approach is the basis for pulling the Mn/ROAD research findings together into a mechanistic-based design procedure and calibrating it for conditions similar to Mn/ROAD's.

The nature of a mechanistic design procedure based on Mn/ROAD research will depend on the distress experienced by the Mn/ROAD test sections and the basic relationships uncovered under the research program. The research program will start with simpler pavement load response models and build toward more complex ones. For instance, static load models will be addressed before dynamic models. Elastic-, inelastic-, and viscoelastic-based models will be evaluated. Analytical models such as VESYS as well as numerical models such as finite-element models will be evaluated. Climatic effects models such as FHWA's integrated model are also an important part of a mechanistic design procedure and will be evaluated and calibrated also.

Two key (difficult) components of such a design procedure are variability and the transfer functions. To understand the effects of variability on the reliability of a design procedure, it is important to have a good understanding of the variability of all the data and findings that feed into the procedure. Transfer functions ultimately relate information from field measurements about pavement distress and performance to model predictions. For a mechanistic-based model, the transfer function converts an accumulation of short-term pavement responses to longer-term pavement distress or performance. Such links have historically been the weak point of pavement design procedures.

Finally, to counteract the popular myth that the research is done when the report is written, two other Mn/ROAD objectives are to support the implementation of the research findings throughout the life of the project and to support the education of a new generation of pavement experts. Although the Mn/ROAD research program is expected to last 20 years, preliminary findings and even final reports will be forthcoming throughout the life of the project. This means that the development of pavement engineering expertise through education and information exchange, as well as implementation of tools for pavement structure practitioners, will be an ongoing process in the Mn/ROAD research program.

This is a broad overview of the Mn/ROAD facility, its design and purpose, the data that will be generated there, and its potential for supporting significant pavement research. More detailed information and data are available by visiting the Mn/ROAD site and through a number of available handouts, summaries, and reports.

ACKNOWLEDGMENTS

Matthew W. Witczak of Law Engineering, Inc. and the University of Maryland provided valuable direction to the development of a framework for the research program during the design phase of the Mn/ROAD facility. More recently, David E. Newcomb of the University of Minnesota and George R. Cochran of Mn/DOT were instrumental in fleshing out the program into a logical and viable entity. Nelrae Succio of Mn/DOT and others too numerous to mention played significant roles in the development of the Mn/ROAD facility.

REFERENCES

1. Kulash, D. *FOCUS*. Dec. 1993, p. 6.
2. McCullough, F. Paper presented at Pavement Testing Workshop, Austin, Tex., July 1992.
3. *Mn/ROAD Updated Research Management Plan*. Mn/DOT, March 1994, p. 1.
4. Daleiden, J. Evaluation of the AASHTO Flexible Pavement Design Equation Utilizing the SHRP LTPP Data Base. Presented at the 72nd Annual Meeting of the Transportation Research Board, Jan. 1993.
5. Baladi, G. Y. Mechanistic Evaluation of the AASHTO Flexible Pavement Design Equations. In *Transportation Research Record 1449*, TRB, National Research Council, Washington, D.C., 1994.
6. Calibrated Mechanistic Structural Analysis Procedures for Pavements, Phase 2. NCHRP Project 1-26. TRB, National Research Council, Washington, D.C., Dec. 1992.

Finnish Cold-Mix Asphalt Pavement

Jyri Mustonen and Raimo Tapio, *Finnish National Road Administration*
Sven-Åke Blomberg, *Turku Road District, Finland*

A Finnish-Minnesotan cooperative project to test paving materials for low-volume roads will embark in 1995. The subject of research is a Finnish cold-mix asphalt, oil gravel (*öljysora*). A test road selected by the Minnesota Department of Transportation will be surfaced with oil gravel to judge its suitability for conditions in Minnesota. The contents and mixing requirements of oil gravel and emulsion gravel pavement—aggregate, binder, mixing, and surfacing techniques—are identified on the basis of technical experience with this type of paving material in Finland.

Finnish experience with oil gravel has a long history. The Finnish National Road Administration (FinnRA) has a 77 000-km public road network. Of these 77 000 km some 47 000 km is paved roads—16 000 km with asphalt concrete and 31 000 km with oil gravel. The first modern cold-mix pavements were introduced at the beginning of the 1960s. The lifespan of an oil gravel pavement can be up to 20 years. Oil gravel and recently developed emulsion gravel are very similar products, the main difference being the binder.

Applications

Both oil gravel and emulsion gravel are used as surfacing for roads where average daily traffic (ADT) is at most 1,000 vehicles per day. Oil gravel is also used as patching or surface treatment mix for oil gravel roads.

The price of the oil gravel or emulsion gravel wearing course is about 65 percent of the price of an asphalt concrete wearing course. Annual statistics show that, for example, in 1993 the average price of asphalt concrete (100 kg/m^2) in Finland was 12.22 mk/m^2. In the same year, the average price of oil gravel (100 kg/m^2) was 7.70 mk/m^2.

Layers under the surface are also cheaper in emulsion gravel roads than in asphalt concrete roads: the latest calculations demonstrate that the unit cost of building an asphalt concrete road is 1.3 to 1.7 times higher compared with the cost of a similar emulsion gravel road.

Annual maintenance costs of emulsion or oil gravel roads are closer to those of gravel roads than asphalt concrete roads: according to the latest (1993) maintenance statistics, the average maintenance cost of asphalt concrete was 18 967 mk/km/year. For oil gravel the cost was 10 165 mk/km/year, and for gravel it was 8352 mk/km/year on average. These costs naturally depend on the standard of each road class and are not directly comparable.

The typical thickness of an emulsion gravel pavement layer is 70 to 100 kg/m^2. Emulsion gravel roads are usually designed to last 8×10^5 equivalent single-axle loads (ESALs) during 15 years (10-ton axle).

The number following the surfacing-type abbreviation is the average weight per square meter (kg/m^2) of the surfacing. For example, OS 20/100 means that the maximum grain size of the aggregate is 20 mm and the amount of oil gravel is 100 kg/m^2.

Recycled mixes are marked by adding the letter R (*R-rouhe-crush*) and a number, which shows the percentage of old mix in the total of the mix, to the end of the normal abbreviation, for example, AB 20/120 R 70.

Raw Materials

Aggregate

The aggregate is macadam or crushed gravel containing fine aggregate. The mixing properties and the grading of the mix can be improved by dividing the aggregate into grading classes. If grading, cleanliness or other properties are not in accordance with quality control, or if there are fears that the contents of the aggregate may essentially decrease the quality of the oil gravel, the success must be verified beforehand with laboratory tests.

The quality of crushed stone produced with the blasting and crushing methods used in the manufacture of surfacing aggregate must be as high as possible. The quality of aggregate used for surfacing must be tested in an approved laboratory.

All crushed stone used for paving must meet the quality requirements for the following upper and lower grading limits. The particle size fraction may not contain more than 5 percent by weight of material coarser than the upper grading limit. The whole particle size fraction must pass a sieve size 20 percent larger than the upper grading limit. The particle size fraction cut from both ends may not contain more than 10 percent by weight of material finer than the lower limit. At most 4 percent by weight may pass the sieve that is half smaller than the lower limit. If the particle size fraction has been determined with wet sieving, it may contain at most 2 percent by weight of material passing the 0.074-mm sieve. The latter does not concern such particle size fractions whose lower limit is zero.

Aggregate may not contain harmful quantities of contaminants like clay, peat, humus, topsoil, wood, ice, or salt. During stocking the aggregate sorts may not segregate nor mix together nor with the soil under them. The humus class of the aggregate can be determined by the NaOH test (TIE 221 method). For aggregate used for cold paving the maximum organic material (humus) class is II. If the humus class is III or worse, the qualifications of the aggregate must be determined separately by doing adhesion tests for test mixes, among others. Finnish TIE methods are in most cases comparable to corresponding ASTM and DIN methods. In the TIE 221 method, classes are from 0 (best) to IV (worse). Classes 0, I and II are proper without specific examinations. The cleanliness of fine aggregate is especially important in cold-mixing techniques.

The following properties are determined from the aggregate:

- Point load index Is(50) (determined from a drill core),
- Ball mill value,
- Abrasion value (before ball mill value), and
- Shape index and flakiness.

The strength classification is shown in Table 1. The strength class is determined by the weakest value.

The shape classification of the aggregate is shown in Table 2. The shape class is determined by the shape index or the flakiness, depending on which one of them is worse. The shape class determinations are normally done using aggregate from the grading Class 8 to 12 mm. If the crushed stone product does not contain grain sizes 8 and 12 mm, the shape and quality class are determined using the grading Class 12 to 16 mm.

The aggregate of emulsion gravel is crushed stone similar to that of the oil gravel aggregate. The highest permissible humus content class is II. If the humus content class is higher than Class II, the suitability of the aggregate to the manufacture of emulsion gravel can be determined beforehand with laboratory tests.

Binder

In oil gravel the binder is BO-2T, road oil with adhesion-improving additive. The amount of adhesion additive must be high enough to ensure active adhesion. The adhesion-improving additive is a mix of mono- and diamines or just diamine. The recommended percentages for adhesion-improving additive are as follows:

Adhesion-Improving Additive	*Undrained Aggregate (%)*	*Drained or heated aggregate (%)*
Mono- and diamine mix	1.2	0.5
Diamine	0.8	0.5

TABLE 1 Strength Classes of Aggregate

Class	Point Load Index Is(50) TIE 241 MPa	Ball Mill Abrasion Value TIE 242 %
IA	≥ 13	≤ 7
IB	≥ 10	≤ 11
IC	≥ 8	≤ 14
ID	≥ 6	≤ 17
II	≥ 4	≤ 30

TABLE 2 Shape Classes of Crushed Stone and Their Limits

Shape class	Shape Value TIE 233				CEN-Draft prEN 933-6:1992 Flakiness %
	Elongation (c/a) Researched Fraction (mm)		Flakiness (b/a) Researched Fraction (mm)		
	8 - 12	12 - 16	8 - 12	12 - 16	
I	≤ 2.5	≤ 2.3	≤ 1.5	≤ 1.4	≤ 10
II	≤ 2.6	≤ 2.4	≤ 1.7	≤ 1.6	≤ 15
III	≤ 2.7	≤ 2.5	≤ 1.8	≤ 1.7	≤ 20
IV	≤ 2.9	≤ 2.7	≤ 1.9	≤ 1.8	≤ 25

If the additive is added to the binder in a refinery, the values are 0.1 percent higher than just given in the table. If adhesion between road oil and aggregate is not good, the additive content can, if necessary, be 0.1 to 0.3 percent greater.

The melting of the adhesion-improving additive is done on site in special melting kettles or by mixing it directly with the binder. The binder tanks must have efficient mixing equipment. After the additive is completely mixed with the binder, the mixing process must be continued until the mix has once more gone through the pump.

Gradually, mainly because of oxidation, the quantity and effect of the adhesion-improving additive mixed with the binder decrease. As the temperature increases the oxidation also increases. To reduce oxidation, the handling of the binder-adhesion mixture must be organized to make circulation in the tank possible only below the fluid level, to prevent oxidation increase caused by surging. Safety instructions for the handling of adhesion should be followed. Both the quality and the quantity of the adhesion and the operation time for the binder after reducing the adhesion are specified in the contract.

In emulsion gravel, the binder is of bitumen emulsion BE 1000 to 3000. The stiffness of the binder is chosen according to its use and desirable final strength of surfacing. Binder can be emulsified in the refinery or in the mixing plant. The emulsification of the binder in the mixing plant requires the emulsifying equipment to be connected with the mixing plant. Other specific requirements for bituminous-oil and bitumen-emulsion binders are given in Table 3.

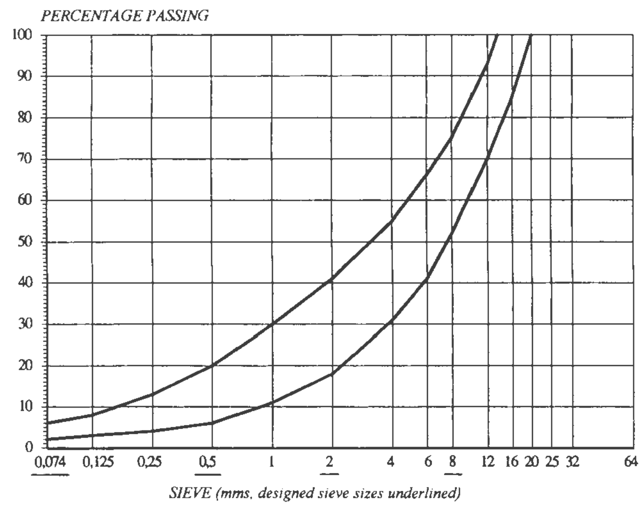
Manufacture of Mix

Figure 1 shows the general regulations for the composition of oil gravel mix and for the quantity of mix in the completed surfacing. Unless otherwise indicated, the mixture is made of undrained aggregate. Drainage and heating of the aggregate are usually necessary if

TABLE 3 Requirements for Bituminous Oil and Bitumen Emulsion

Aim of Testing	Requirement	Unit	Bituminous Oil BO-2	Bitumen Emulsion BE
Viscosity 25 oC	min.-max.	mm2/s		35-170
Viscosity 60 oC	min.-max.	mm2/s	350-650	
Fractional distillation				
- Distillate of the initial amount up to 225 oC	max.	vol %	0	
- Distillate of the initial amount up to 260 oC	max.	vol %	1.0	2.0
- Distillate of the initial amount up to 315 oC	max.	vol %	8.0	
- Distillate of the initial amount up to 360 oC	max.	vol %	12.0	
Distillation residue 260 oC	min.	weight %		60
Viscosity of distillation residue 60 oC	min.	mm2/s	2 000	900
Viscosity of distillation residue 60 oC	max.	mm2/s	4 000	4 500
Distillation residue, solubility to toluene	min.	weight %		99.5
Water	max.	weight %	0.5	
Flash point	min.	oC	56	
Sieving residue on 0.5 mm sieve	max.	weight %		0.2
Sedimentation, 5 days	max.	weight %		4
Breaking	max.	weight %		60

MINERAL AGGREGATE		
Crushed aggregate		0 to 12, 0 to 16 or 0 to 20 mm
BINDER		
OG:	BO-2	3.2 to 3.6 weight %
EG:	BE-ES	3.2 to 3.6 weight % (retained binder)
ADDITIVE		
Anti-stripping agent OG:	a mixture of mono to and diamine	0.8 to 1.3 weight % approx. 0.8 weight %
Anti-stripping agent EG:	diamine	approx. 0.5 weight % (of the weight of the binder)



Amount of asphalt mixture in an asphalt slab of standard thickness 70 to 100 kg/m²

FIGURE 1 General regulations for the composition of oil gravel.

1. The percentage by mass passing a 0.074-mm sieve is more than 5 and the moisture content is over 3 percent,
2. The moisture content is more than 5 percent,
3. The air temperature during the laying mix is below +5°C and below 0°C if the mix is manufactured for stocking, and
4. The percentage by mass passing a 0.074-mm sieve is above 6.

The moisture content of drained and heated aggregate should usually be between 0.7 and 1.5 percent. The separation of aggregate and mixture should be prevented.

The spraying temperature of the road oil must be 100 ± 15°C before mixing with the aggregate. The mixing temperature is shown in Table 4.

In cold feeding process the feeding of aggregate and binder into the mixer should be done together and the feeding should last as long as possible; this way the mixing is effective. If the mixing time has been long enough, the mixture is homogenous and without clods. Oil gravel is not to be mixed on the road.

TABLE 4 Permissible Mixing Temperatures of Soft Mixture

Mixture	Equipment	Binder	Mixing Temperature °C
Soft asphalt concrete	Asphalt plant	B-800	110 - 130
Soft asphalt concrete	Oil gravel plant + drum or steam heating	B-800	60 - 90
Recycled soft asphalt concrete	Recycling plant	BÖ-4 B-800	70 - 100
Oil gravel	Oil gravel plant	BÖ-2	Cold mixing
Oil gravel (heated)	Oil gravel plant + drum or steam heating	BÖ-2	40 - 70
Oil gravel (dried)	Asphalt plant	BÖ-2	80 - 100
Recycled oil gravel	Recycling plant	BÖ-2	50 - 80

Before paving, the mix composition must be ascertained with weighing machines and checked with meters. The checking can also be done by taking a sample from the trial heat and testing it in the laboratory. Paving may start if the results show that the mix fulfills the requirements and if the composition (clods, segregations, etc.) and other properties (color, adhesion) are considered normal. The contract price can be reasonably reduced if the material laid on the road is not adequate. If failure makes the surfacing essentially worse, it must be totally removed. The mixing time and capacity are agreed upon separately for each site after it has been ascertained that the mix meets quality control standards.

The quality of cold mix can be improved by stocking. The usual amount of oil gravel stocked for maintenance is 30 tonnes per kilometer. Stored oil gravel usually contains 0.2 percent more binder than oil gravels used immediately after their manufacture. Oil gravel should not be stored in rainy weather if increasing moisture content decreases the quality of the mix. The disparition is to be avoided. The stockpiles should be made consistent to protect the mix from becoming dirty and getting damp.

Emulsion gravel mix is manufactured according to the instructions for oil gravel, to a certain extent. Emulsion gravel can be manufactured cold or heated. The best mixing results are achieved when the moisture content of mix is 2 to 3 percent.

Surfacing

The mix is usually laid with an asphalt paver so that no hand-laying is needed, except joint sealing. If so decided, the laying can be done with a drag or with other equipment suitable for this purpose. The amount of mix used and the area paved are noted after each work shift.

The average material consumption during each shift must be at least the same as the quantity ordered. The material must be laid evenly along the surface. The quantity of the mix measured in one place may at most be 15 percent smaller than ordered. The quantity of the material laid can be observed by comparing the weight of each truck with the corresponding area. Thin places must be corrected during the work.

The oil gravel is usually rolled soon after laying. Attention should be paid to the compaction of the edges of the road. Traffic on the new oil gravel road should be limited if the traffic loads seem to cause rutting, binder bleeding, or other serious damage.

The reasons for the weak adhesion between the aggregate and the binder, for the significant unsticking of large stones, for the binder bleeding, and for other similar failures of the new oil gravel surfacing must be determined and eliminated. If necessary, the paving work must be interrupted during the reconditioning.

If there are smooth or soft places or if the moisture content of the prepared surfacing is too high, the surface must be milled. After milling there should be a 2.5-cm-high prepared layer on the pavement. The surfacing is rolled after milling.

Emulsion gravel is laid with an asphalt paver and compacted efficiently in the same way as oil gravel. Warm mix is more easily workable than cold mix. Because the strength development of emulsion gravel is rapid, the mix must be compacted immediately after laying and the surfacing needs more compaction than oil gravel.

Milling and Increase of Mix

Old oil gravel surfacing is generally repaired by milling the underlay, increasing the mix, and compacting. Old oil gravel is milled by a ripping/milling device joined to a motor grader or by a separate miller.

Milling can be done only in dry weather. To avoid clods the surfacing can be warmed. Milling must not be done in such a way that the crushed stone below it is mixed with oil gravel. After milling the underlay is leveled. New mix is added to the leveled underlay and the surfacing is compacted.

The design grading curve must be chosen by the use. After the design grading curve has been chosen, the binder content used for minor works can be determined by interpolating the binder contents of the design grading curves.

If the specific gravity of the aggregate differs from the value 2.70 kg/dm^3 the binder content has to be changed as follows: if the specific gravity of aggregate changes +0.08 kg/dm^3, it is correspondent with the binder content change −0.1 percent by weight.

Recycled Oil Gravel and Soft asphalt Mix

Applications

Recycled oil gravel containing at most 70 percent oil-gravel crush is used as the normal oil gravel. The recycled oil gravel in which the oil gravel percentage is over 70 is used for low-traffic roads (with AADT less than 300 vehicles per day).

The applications for the recycled soft asphalt are the same as for the normal soft asphalt concrete. The manufacture and the laying are done following the instructions for the recycled oil gravel.

Oil-Gravel Crush

The old oil gravel surfacing is milled with an asphalt miller or a scarifier with a motor of its own. These machines have adjustable milling depth and the crush is loaded directly onto the vehicle.

The oil gravel surfacing suitable for milling should be even, so that milled crush will not contain too many big stones and crushed aggregate from the base. The surfacing should be solid and coherent enough to bear milling without splitting into clods. It is better not to mill the edges of the surfacing or the wide and puddled failures; if these are milled, the crush must be stored separately. Oversized grains must be separated from the crush with sieving.

The serviceability of the milled part of the road can be improved by leaving a sufficient unmilled layer on top of the underlay. The milling should preferably be done just before the improvement is started, since holes appear easily in the milled base.

The oil gravel crush is stored in loose stacks. The machines may not move on top of the stacks. If the crush is stored over the winter, the stacks should be covered.

The maximum grain size for the oil gravel crush is typically 90 percent of the original maximum grain size. The quantity of the fine aggregate (percentage by mass passing the 0.074-mm sieve) is 7 to 9 percent. The binder content varies between 2 and 3 percent.

Proportioning

The grading of the recycled oil gravel can be improved with the crushed stone containing little fine aggregate, for example, grading Class 8 to 18 mm. In general the aggregate increase is over 30 percent.

The additional aggregate is often used on busy roads, where it increases the quantity of active binder in the mix and decreases the tendency for segregation.

The crush stimulated with nothing but additional binder can be used on low-traffic roads, although such a mixture tends to be dry and to segregate.

The quantity of additional binder is calculated with the following formula:

$$P_{add} = P_{aim} - 0.85 \ldots 0.95 * R/100 * P_{crush}$$

where

P_{aim} = binder content determined by the proportioning,
P_{crush} = binder content of the oil gravel crush, and
R = oil gravel percentage in recycled mixture (0 to 100 percent).

The coefficient is 0.85 if the crush contains old and hard binder, and the mix contains a lot of crush (≥70 percent). The coefficient is 0.95 if the crush is moist, and the crush content of the mix is small (≤50 percent).

The binder used is made of road oil BO-2 cured with adhesion-improving additive. When oil gravel is used in the manufacture of soft asphalt concrete, the binder used is made of road oil BO-4 or bitumen B-800.

Manufacture and Laying of Mix

The manufacture and laying of mix are done according to the regulations for normal oil gravel. The cold-mixing can be done either in the batch-mixing or continuous-mixing oil gravel plants. In the continuous mixing process the functioning of the cold feed unit must be controlled all the time since the feeder tends to be clogged rather easily.

The heat mixing can be done either in a batch-mixing plant supplied with a heating drum and a batch mixer or in an asphalt plant with a continuous drum mixer. In the continuous mixing plant the cold feeder unit must have an automatic belt weighing machine for aggregate weighing. The final moisture content of the mix has to be 0.4 to 0.8 percent.

Quality Control

Good working methods and building materials as well as skilled foremen and workers should be used. One sample per 500 tons of mix is taken randomly from the oil gravel crush and the reclaimed asphalt mix during the crushing and storing of crushed material. The binder content, the grading, and the water ratio are determined from the samples. In order to find out the binder qualities of the crush, one crush sample (about 4 kg) for each beginning 5,000 tons is sent to the central laboratory of the Finnish National Road Administration.

One mix sample is generally taken for each beginning 500 tons of mix. Depending on the maximum grain size, the quantity of the sample to be investigated is as follows:

Maximum Grain Size (mm)	*Sample (g)*
12–20	1,700
>20	1,900

The samples are investigated to determine their binder content and grading and also the water ratio of oil gravel mixes. The test results must be ready by the time about 500 additional tons of mix have been manufactured (since the samples were taken). The sampling systems for projects that need a small increase of mix (remixing, ART, etc.) are agreed on separately.

The average composition of the mix is determined from the average samples. The quantity of one hot-mix sample is about 8 kg and the quantity of one cold-mix sample is about 7 kg.

The adhesion of oil gravel is always investigated after the increase of adhesion-improving additive and randomly at an interval of 500 tons of mix (the bucket test). If the adhesion is poor, it must be tested. Cold-mixed oil gravel has to be checked at an interval of 5,000 tons of mix. At least one adhesion control test is taken from every mixing plant.

For aggregate and mixture weighing the continuous oil gravel mixers must have weighing machines equipped with belt scales with an accuracy of ±2.0 percent. For oil quantity measuring there must be flow meters with an accuracy of +1.0 percent. The binder content measured with such a meter may not differ more than +0.2 percent units from the design value.

Oil gravel or soft asphalt concrete joints are not heated or coated. Recycled oil gravel or soft asphalt concrete must meet the quality control standards for normal oil gravel or soft asphalt concrete. The quality control standards for emulsion gravel is the same as those for oil gravel.

Conclusions

Finnish oil gravel will soon be tested in the United States by the Minnesota Department of Transportation (MnDOT). In Finland and other Nordic countries, especially Sweden, oil gravel pavement has taken its place as a surface type for low-volume roads. The price-quality ratio is best for this pavement type when the traffic volumes are less than 1,000 vehicles per day. The adaptability of this pavement type to Minnesotan climate and traffic conditions will be apparent after one year of traffic load on the pavement. So far, the cooperation between MnDOT and FinnRA has been a success story in many areas of road and traffic engineering, including winter maintenance, an engineer exchange program, and the MnROAD project. The joint project on oil gravel pavement should also be among the successful cooperation projects between these two road administrations.

REFERENCES

1. Emulsiopäällysteiden suunnittelu ja rakentaminen (in Finnish). *Tielaitoksen selvityksiä* 82/1993. Helsinki, Finland, 1993.
2. *Finnish Asphalt Standards* (in test use 1994). Finnish Pavement Technology Advisory Council, Helsinki, 1994.
3. *Paving Works* (Tienrakennustöiden yleiset laatuvaatimukset ja työselitykset) In Finnish. FinnRA. Helsinki, 1989.
4. Räme Raimo. *Öljysorapäällysteet* (in Finnish). Neste Oy, Porvoo, Finland, 1989.
5. *Tie- ja siltapäällysteiden urakkahinnat vuonna 1993* (in Finnish). Tielaitos, Kehittämiskeskus, Helsinki, 1993.
6. Yleisten teiden kunnossapitotilasto (in Finnish). *Tielaitoksen tilastoja* 2/1994. Helsinki, Finland, 1994.

Cold In-Place Recycling on Low-Volume Roads in Austria

Johann Litzka, *Vienna University of Technology, Austria*
Wolfgang Haslehner, *County Government of Burgenland, Austria*

Modern methods for road maintenance should involve used construction materials, take account of environmental compatibility, and eliminate road damage economically and durably. Regarding these basic requirements, attention should be paid to cold in-place recycling of damaged asphalt layers using cement stabilization. Within the last few years, cold in-place recycling has become an appropriate alternative for the rehabilitation of low-volume roads in Austria. In the course of documentation carried out at the Institute for Traffic and Transportation Engineering of the Vienna University of Bodenkultur, the individual steps of construction were analyzed. The advantage of the described procedure is that none of the old pavement need be hauled to a special repository. An innovative method for cold in-place recycling on low-volume roads using cement stabilization is described. The first step of this method contains a detailed analysis of the section to be restored, including bearing capacity measurements and the determination of the grading curves of existing unbound layers. Grading curves are also determined for the existing asphalt layer after trial milling in order to consider refinement by milling. This analysis forms the basis for adding material before milling in order to achieve a well-graded aggregate. On the construction site, the necessary additional aggregate is spread over the existing pavement. In the next step, the cement binder is distributed on the road surface. A soil stabilizer breaks up the existing road structure and mixes it thoroughly with the aggregates and binder that were distributed beforehand. The leveling and compaction are done with a grader and a vibratory roller. An after-treatment with a vibratory roller is carried out up to 3 days later to create microcracks and prevent the appearance of open stress-caused cracks at greater intervals. The paper also describes in detail the analysis of existing pavement before milling, including bearing capacity measurements. Existing results and statements describing achievable quality level are given. Results showing increased and homogenous bearing capacity conditions after milling and cement stabilization are shown. Concluding remarks also show that this way of reconstruction is an economic means of conserving raw materials and protecting the environment.

The low-volume rural road network in Austria consists of all roads that are neither federal nor provincial and that render access to the rural areas of the country. This low-volume road network consists of all local roads outside the villages, farm roads and forest roads. Its total length amounts to about 160 000 km (approximately 99,300 mi). Traffic volume on these rural roads amounts to a maximum of 250 vehicles per day on an annual average.

Since the responsibility for construction and maintenance of this road network is split up between private persons and public authorities, there is no exact documentation on the condition and length of these roads. In practice until now, maintenance was neglected in many cases and adequate new reconstruction techniques have been increasingly needed.

Within the last few years, cold in-place recycling has become an appropriate alternative for reconstruction of low-volume roads in Austria. The advantage of the described innovative method is that all of the existing material can be used within the new construction. Many tests during the last 10 years were carried out with cold in-place recycling using bituminous stabilization and cement stabilization as well. Especially for rural roads with normally poor bearing capacity and poor-quality existing pavement material, cement stabilization was found to be the more appropriate solution.

In the course of documentation carried out at the Institute for Traffic and Transportation Engineering of the Vienna University of Bodenkultur, each step of construction was analyzed critically (*1*). This forms the basis for the further improved application of this construction method.

Construction Procedure

Cement stabilizations mixed in-plant have been used for reconstruction and overlays on rural roads since 1984 (*2*). These experiences form the basis for use of cement stabilization with the in-place recycling method at the beginning of the nineties. In summer 1993, this construction method was applied at seven sites using the experience of the last several years. At these sites the construction process was documented extensively (*1*) in order to describe the random conditions, preconditions and special requirements for the use of this construction method. The total length of these sites is 5.88 km, with an average width of 3 m. Thus the treated area is approximately 17 600 m^2.

General Remarks

The described method for cold in-place recycling can generally be divided into the analysis of the existing pavement, the adding of necessary unbound material, the distribution of cement binder on the surface, the milling by means of a soil stabilizer, the leveling with grader, the compacting with vibratory rollers and the after-treatment procedure.

Within this method for cold in-place recycling, the unbound subbase layer and the wearing course are milled together with added material and portland cement binder, resulting in a cement-stabilized layer. An additional surface layer is necessary. In most cases a bituminous surface dressing will be enough. Very often asphalt concrete surface layers (with a thickness of about 40 to 60 mm) are used.

The following advantages and benefits can be achieved by using the presented method:

- Economical and durable elimination of damage;
- Increased and homogenous bearing capacity;
- Frost sustainability;
- Total recycling of existing subbase and wearing course material;
- No waste material for deposit;
- Minimized environmental impacts, including no use of raw materials, minimal use of resources such as gravel, and little or no additional transportation of materials; and
- Brief traffic interruptions during construction.

Preliminary Examinations

Thickness Investigation

According to a design catalogue for rural roads in Austria (*3*) the necessary thickness of the cement-treated layer can be determined in relation to the existing design traffic. A preliminary investigation of the thickness of the existing pavement must show that it is possible to reach the necessary thickness of the cement treated layer with sufficient certainty. If the existing pavement shows insufficient thickness it is necessary to place additional unbound material (natural gravel or reclaimed granular asphalt) on the road surface before milling. The choice of this method should be made taking into account the aspects of the optimum grading curve shown in the next section.

Unbound Layer Examination

In general, subbase materials of existing minor rural roads in Austria with an age of approximately 30 years show grading curves with a high percentage of material less than 0.02 mm, thus showing a rather high frost susceptibility; and maximum particle size between 32 and 56 mm, amounting to between 5 and 10 percent by mass.

In most cases it is necessary to make some corrections of the grading curve of the existing material before stabilization to achieve a good result. The necessary preceding investigation must consider not only the existing unbound base material, but the material that results from the milling process, including base material and milled asphalt material. This is the reason why this investigation should be made after a trial milling on site. In cases where such a trial milling is not possible for organizational and economic reasons, the investigation of the unbound layer should be made after adding a respective amount of reclaimed granular asphalt in the laboratory.

The aim of the correction of the grading curve is a gradation to reach a good stability and compactability of the layer. An orientation for this gradation can be derived from the grading curves for unbound base layer materials as given in the respective Austrian specification (*4*), shown in Figure 1.

For cement stabilization, it is not necessary to restrict the content of particles smaller than 0.02 mm to 3 percent by weight, which also means that slightly frost-susceptible material can be used. With respect to the following treatment with cement, it is possible to allow higher amounts of fines (< 0.02 mm). But it has to be taken into account that a content > 10 percent results in a rather high necessary cement content to reach frost sustainability of the cement aggregate mixture (see section entitled "Suitability Test"). This high cement content normally leads to a high strength of material and increases the possibility of shrinkage cracking.

Another point is the sand content (0.06 to 2 mm) because these sand particles are of great importance for the mortar filling the voids between the bigger particles and increasing the stability of the mixture. If the sand content is lower than 10 percent, it is necessary to add sand to avoid too coarse an aggregate.

Suitability Test

With the unbound materials composed along the lines of the previous section, the suitability test must be performed in the laboratory to find the appropriate mix design, that is, the necessary cement content.

This test follows the Austrian specification RVS 8.05.13 (*5*), taking into account two different requirements, compressive strength and frost sustainability.

For these tests, specimens with different cement contents compacted at optimum water content are made using a proctor cylinder. These specimens are damp-storaged for 7 days at a temperature of +20°C. Compressive strength after this time should not be less than 3 MPa. Experiences also show that this value should not exceed 5 MPa because higher compressive strength brings much higher risks of wide reflective cracking (package cracking).

Frost susceptibility testing, which also starts after these 7 days, is carried out according to the ASTM Method D 560-57. The height difference between the first and twelfth frost cycle must not exceed 1 percent.

The minimum cement content is normally fixed with 90 kg/m^3 of compacted mixture. In cases of coarse aggregate this value could create compressive strength that is too high. This limit can therefore be undercut if necessary.

Construction

Stabilization Procedure

Before milling is performed, the additional unbound material, if necessary, is spread over the surface of the existing pavement that needs reconstruction. The cement binder is distributed by means of a spreader immediately before milling.

In the next construction step, a soil stabilizer breaks up the existing old road surface as well as the subbase

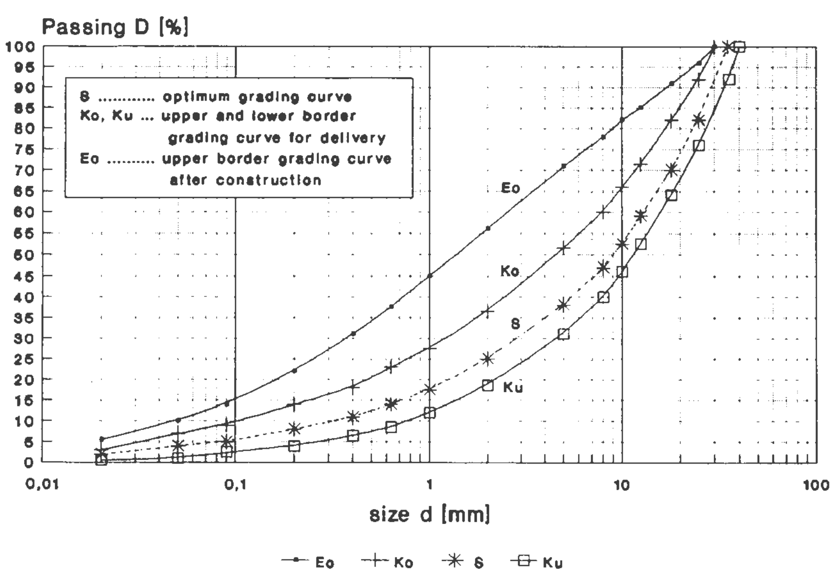


FIGURE 1 Grading curves for base material, grading 0/35 mm (*4*).

layer and mixes it thoroughly with the aggregates and the cement binder distributed beforehand. Practice has shown that a maximum milling depth of about 30 cm can be achieved.

A truck with a tank is connected to the soil stabilizer to provide the necessary water for the mixing process. The necessary moisture content for setting and compaction is achieved through a batching unit in combination with an injection beam integrated in the soil stabilizer. A moisture content of about 6 percent of the milled material has proven ideal.

For control purposes it is advisable to take samples of the fresh mixture to analyze the effective gradation at greater intervals. A comparison with the grading curve used at the suitability test could sometimes lead to small corrections of the mixture design.

After milling and mixing, the material is leveled by a grader, considering the projected necessary transversal gradient. Compaction is conducted by 100 kN-vibratory steel rollers with a small amplitude. Following the respective Austrian specifications, the compaction rate (dry density of the layer versus proctor dry density) has to be $\geq$ 100 percent on average with no value < 97 percent in the whole section.

To prevent the stabilized layer from drying out too fast, the surface is kept wet for between 3 and 5 days according to climatic conditions. Another possibility is to place a bituminous surface dressing (chipping size $^2/_4$ or $^3/_5$ mm) soon after construction of the stabilized layer. This brings the additional advantage of better bond between the cement stabilized base and the surface course placed later.

Immediately after construction the new pavement can be opened for slight traffic. Truck traffic is normally permitted 24 hours after construction.

Aftertreatment

As a result of tensile stress caused by shrinkage during setting and changes of temperature, transverse cracks often occur at regular intervals of about 15 to 20 m. To avoid or minimize the development of shrinkage cracking, the compressive strength is limited at the design process (see section entitled "Preliminary Examinations"). Furthermore a special procedure for aftertreatment has been developed and applied successfully.

The cement stabilized layer is loaded through several passes with vibratory rollers after a timespan of between 24 and 72 hr, thus creating a microcrack structure in the stabilized layer.

Practical experiences show that five roller passings lead to satisfactory results and the created microcrack structure prevents the development of larger stress-caused cracks successfully. Thus reflective cracking in overlying asphalt courses does not appear.

Measurements on Test Sites

In addition to the routine investigations and tests necessary at test sections, bearing capacity measurements before and after construction were made. These measurements were carried out with a modified Benkelman beam with automatic data collection under a wheel load of 50 kN.

Figure 2 illustrates a series of deflection diagrams measured on one of the test sections at various times:

- On the existing pavement before reconstruction,
- After reconstruction (stabilization) and before microcrack creation,
- After microcrack structure creation, and
- Three weeks after construction.

Table 1 shows as a result of statistical calculations the mean values, the standard deviations, and the representative deflections (mean value + 1.3 $\times$ standard deviation) for these measurements.

It can generally be stated that increased and homogenized bearing capacity takes place. The mean deflection of the measurement on the existing pavement before milling was 1.63 mm, whereas this mean value is reduced to 0.72 mm 3 weeks after construction. The standard deviation before treatment had a value of 0.55 and 3 weeks later this value has been reduced to 0.27. Accordingly the representative deflection is reduced from 2.35 to 1.07 mm during this time.

A comparison between deflection measurement before and after microcrack initiation shows an increase of the mean values (from 1.09 to 1.32 mm). Nevertheless, this increase of the mean values has been reduced in the course of the setting process 3 weeks later by approximately 50 percent.

In addition to the standard deflection measurement, deflection basins at single measuring points have been determined on all test sections. The deflection basins were determined by measuring the deflections in various distances from the actual loading point using an interval of 30 cm up to a distance of 3 m with a concluding measurement at 4 m distance.

The backcalculation of E-modulus was performed following the method proposed by Yang (6). Backcalculations from measurements conducted approximately 28 days after construction showed the following results for the E-modulus of the cement-stabilized layer:

- Maximum value: 6300 MPa
- Minimum value: 2800 MPa
- Mean value: 4500 MPa

This mean value corresponds very well with the value of 5000 MPa, which is used normally for analytical calculations in Austria (3).

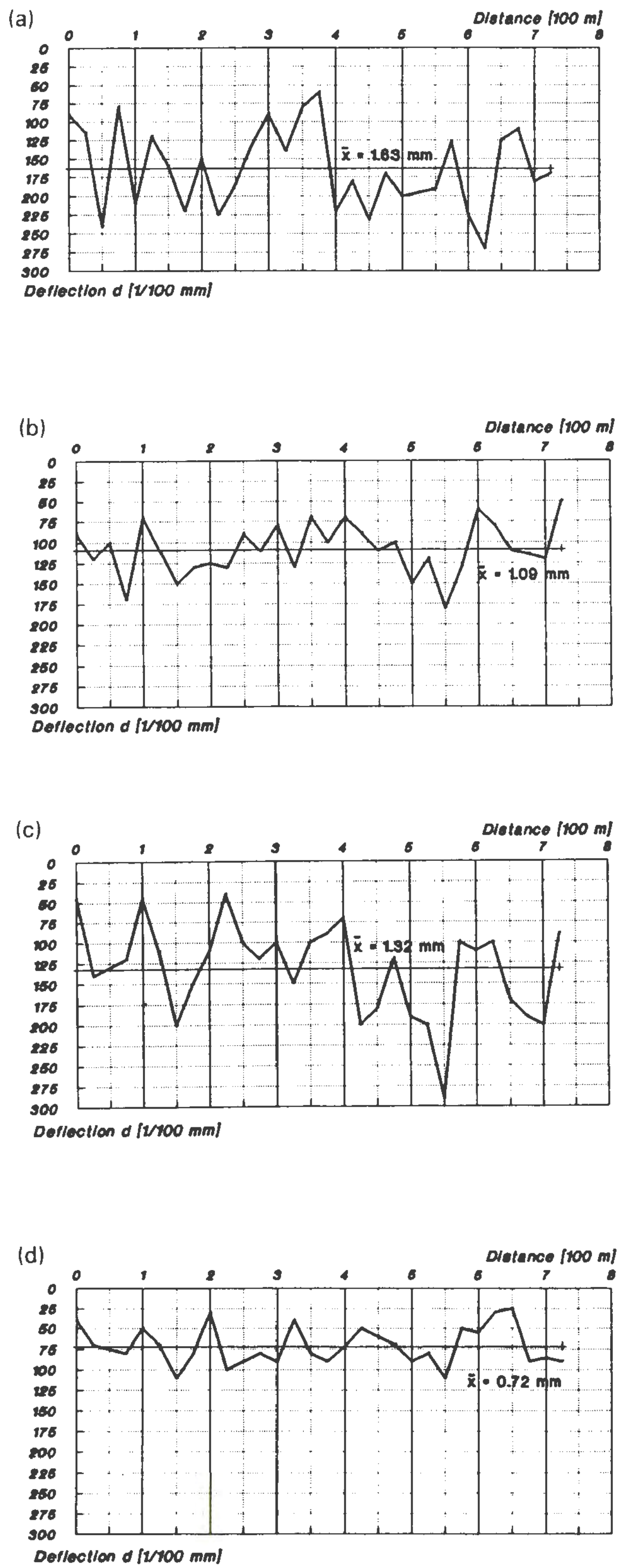


FIGURE 2 Deflection diagrams: (*a*) on existing pavement before reconstruction, (*b*) after reconstruction (stabilization) and before microcrack creation, (*c*) after microcrack structure creation, and (*d*) 3 weeks after construction.

Economic Evaluation

As a basis for economic evaluation of cold in-place recycling under Austrian conditions, a comparison was made with the generally used reconstruction method of excavating the frost-susceptible old pavement down to formation level and placing a new unbound base layer and asphalt wearing course. The cost comparison also accounted for transport and deposit of the excavated material. For this method one must calculate a cost of approximately ATS 250/m^2, whereas cement stabilization mixed in place costs about ATS 150/m^2.

Thus calculations show that costs for cold in-place recycling per square meter of pavement are about 50 to 70 percent of the costs of the traditional method of reconstruction.

These figures only show the range of the possible reduction. One must take into account great differences in local conditions. For instance in cases where the material which must be added before stabilization is very expensive, the application of this cold in-place recycling method can become uneconomical.

General Conditions and Hints for Application

The described method is applicable primarily to old pavements with coarse aggregates. For roads with only thin unbound layers or with very large aggregates this method cannot be used economically.

Furthermore one must take into account that reconstruction by recycling in place is restricted to the width of the existing pavement and thus to cases where this width is sufficient. If widening of the cross section is necessary a total reconstruction is normally better.

Regarding the fact that in most cases additional material is necessary (to improve the grading or to reach the necessary thickness of the stabilized layer) the surface of the new pavement lies at a higher level than that of the old pavement. Therefore this method cannot be used in cases where there are serious height restrictions.

Besides all these technical aspects concerning the pavement itself, it should be pointed out that in connection with reconstruction measures such as recycling in place, an improvement or reconstruction of the drainage facilities also must take place. To pay attention to the pavement alone without improving the drainage conditions in many cases will bring distresses after a short time.

Summary

The presented method for cold in-place recycling using cement stabilization has in practice proven to be a good

TABLE 1 Comparison of Deflection Measurements at Various Times

	a) existing pavement	b) after reconstruction	c) after micro-crack creation	d) after three weeks
mean [mm]	1.63	1.09	1.32	0.72
standard deviation	0.55	0.31	0.56	0.27
representative deflection [mm]	2.35	1.49	2.05	1.07

way for reconstruction of existing heavily damaged pavements on rural roads, provided that the necessary preliminary examinations and laboratory tests for mixture design are performed.

The results of deflection measurements before and after construction illustrate that an increase and homogenizing effect in bearing capacity can be achieved. This makes this method an economical solution to the increasing reconstruction demand, especially on low-volume rural roads. The economic benefit depends to a very high degree on local conditions such as the costs of material, transportation, and materials deposit.

REFERENCES

1. Veits, A. *Fräsrecycling auf ländlichen Straßen in Oberösterreich.* Diplomarbeit am Institut für Verkehrswesen, Universität für Bodenkultur, Vienna, Austria, 1993.
2. Haslehner, W. *Messungen auf Versuchsstrecken mit zementstabilisierter Tragschicht.* Diplomarbeit am Institut für Geotechnik und Verkehrswesen, Universität für Bodenkultur, Vienna, Austria, 1986.
3. Smrzka, B. *Oberbaubemessung ländlicher Straßen.* Diplomarbeit am Institut für Geotechnik und Verkehrswesen, Universität für Bodenkultur, Vienna, Austria, 1989.
4. *RVS 8.511: Oberbauarbeiten (ohne Deckenarbeiten), tragschichten, Ungebundene Tragschichten.* Richtlinien und Vorschriften für den Straßenbau, Österr. Forschungsgesellschaft für das Verkehrs- und Straßenwesen im ÖIAV, gem. mit BMwA, Vienna, Austria, 1981.
5. *RVS 8.05.13: Oberbauarbeiten (ohne Deckenarbeiten), Tragschichten, zementstabilisierte Tragschichten.* Richtlinien und Vorschriften für den Straßenbau, Österr. Forschungsgesellschaft für das Verkehrsund Straßenwesen im ÖIAV, gem. mit BMwA, Vienna, Austria, 1987.
6. Yang, F. L. Messung der Deflektionsmulden—ein Mittel zur Bestimmung der Materialkennwerte. *Straße und Verkehr,* Vol. 7, 1980.

Evaluation of Cold In-Place Recycling in Kansas

Stephen A. Cross and Bala M. Ramaya, *University of Kansas*

Kansas has many miles of thermally cracked roads, primarily in the western half of the state. Rehabilitation with conventional hot-mix asphalt overlays and hot recycling have not yielded the expected service life before existing cracks reflect through the pavement. Since 1986, the Kansas Department of Transportation has used cold in-place recycling (CIR) with an additive of emulsified asphalt as a cost-effective option for rehabilitating thermally cracked low-volume pavements. Field performance of the final product appears to have more variation than desirable, with an expected life of 3 to 5 years. The results of a 2-year study indicate that the material properties of locally available aggregates are poor, which results in the low strength of CIR mixes. In addition, the in-place air voids of the wearing surface were high and had an adverse effect on the performance of CIR mixes. Improvement of aggregate angularity and gradation with additional new aggregates or chemical stabilization of the materials is necessary to markedly improve CIR performance.

Kansas has many miles of thermally cracked roads, primarily in the western half of the state. Distress includes small cracks at intervals of 4.5 to 6 m on thin pavements, and wider cracks with secondary cracking and depressions on thicker pavements.

Conventional hot mix asphalt (HMA) overlays and hot recycling have not given the expected service life before existing cracks reflect through the pavement. Since 1986 the Kansas Department of Transportation (KDOT) has been cold recycling 80 to 160 km of bituminous pavements per year as part of its "substantial maintenance" or "1-R" program. Cold in-place recycling (CIR) has shown to be a cost-effective option for rehabilitating thermally cracked low-volume pavements (fewer than 140 equivalent 80 kN single axle loads per day) in western Kansas. (*1*).

CIR projects are typically milled to a depth of 100 mm, approximately 1 percent asphalt emulsion is added, and the mix recompacted. The asphalt emulsion typically used has evolved from CMS-1 to CMS-150P, a polymer-modified emulsion. The emulsion content is adjusted up to 0.2 percent in the field to satisfy local conditions. The equipment train typically consists of a milling machine capable of milling one lane in a single pass, a screening and crushing unit that removes and crushes the oversize material (greater than 25 mm), and a pugmill that mixes the milled material with the liquid additive and places the material in a windrow. The windrow is picked up by a paver and placed full width across the lane. The material is compacted with a minimum 30 ton pneumatic roller. The specified density has evolved from 95 percent of control strip density, to 95 percent of field-compacted density, to 95 percent of daily field-compacted density. The field-compacted density is obtained by compacting the material from the windrow 50 blows per side with a manual Marshall hammer at 44°C. Compaction is monitored with a nuclear density meter but compaction moisture content is not usually monitored.

CIR is intended as a maintenance treatment and the projects are typically funded entirely with state funds. Therefore, there in not a rigorous overlay thickness design procedure. However, when the projects discussed here were constructed, recommended overlay thicknesses were less than 50 equivalent single-axle loads (ESALs) per lane, seal coat; 50 to 140 ESALs per lane, a 19-mm HMA overlay; greater than 140 ESALs per lane, CIR not recommended. The use of 19-mm HMA overlays has been phased out in favor of 38-mm HMA overlays because of constructability and performance problems. For the most part, the CIR projects seem to have minimum rutting and a life expectancy of 3 to 5 years. However, field performance of the final product has been more variable than desired.

Objective

The objectives of this study were to review CIR in Kansas, identify material properties and construction procedures that affect CIR performance, and provide information necessary to produce CIR mixtures that will perform satisfactorily under the current traffic conditions.

Scope

A 2-year laboratory-and-field study was undertaken to evaluate the performance of CIR. This paper summarizes the significant findings from that 2-year study (2). Eleven pavements ranging in age from 1 to 5 years at the time of sampling and rated from good to poor in terms of overall performance were selected for detailed sampling and testing. The data were analyzed to determine material and mixture properties and construction procedures that affect CIR performance.

Plan of Study and Test Results

To meet the objectives of the study, a three-phase plan of study was developed. Data from KDOT's 1992 pavement condition survey (*3*) were reviewed, field testing of selected sites was performed, and core samples were obtained for laboratory testing. This paper contains the significant findings from the field and laboratory testing program. The complete plan of study, test data, and conclusions can be found in the final report by the authors (2).

General Description of Projects

Thirty-six CIR projects were reviewed during the summer of 1992 to determine the general condition of CIR pavements in Kansas. The primary distress associated with the sites reviewed was reflective cracking with associated pavement depression, spalling, and moisture damage. The rutting encountered on the majority of the pavements appeared to be densification of the CIR with traffic rather than plastic flow of the mixture. Failure of CIR mixes is progressive, with increased rutting and spalling and moisture damage occurring at reflective cracks.

Field Testing

From the above review, 11 pavements were selected for further sampling and testing. Field testing consisted of examining a 152.4-m test section from each of the 11 sites and obtaining 10 to 12 cores of 102 and 152 mm diameters at 30.5-m intervals. Rut depth measurements, recorded to the nearest 1 mm, were obtained using a 2-m straightedge, and the total length of cracking in each test section was determined. A cone penetrometer was used to evaluate the relative strength of the subgrade at one location in each test section. Traffic loadings were determined from the pavement condition survey (*3*). The results of the field measurements, traffic, and location of each test site are demonstrated in Table 1. The test sections consisted of either a 19-mm or 38-mm HMA overlay or a seal coat, 100 to 125 mm of CIR, and anywhere from 100 to 300 mm or more of old hot mix or road mix. The thickness of road mix varied considerably in the test sections.

Laboratory Testing

The cores were returned to the laboratory and their mixture and material properties determined. One core was obtained from the outer wheel path and one from between the wheel paths on 30.5-m intervals in each 152.4-m test section. Moisture damage and uncoated aggregates were apparent in some cores from each site and on the majority of the cores from Sites 3, 9, and 10. The majority of moisture damage was found near existing reflective cracks.

The cores were sawed into their respective pavement layers and the bulk-specific gravity (ASTM D2726) determined for each layer. Due to the high void content of the CIR, difficulty was encountered in accurately determining the bulk-specific gravity of some of the cores. The cores were not dipped in paraffin (ASTM D1188) due to the difficulty of removing the paraffin for further

TABLE 1 Test Site Locations and Field Measurements

SITE	ROUTE	OVERLAY THICKNESS (mm)	SUBJECTIVE RATING	AGE (yrs)	AADT	DAILY 80 kN ESAL's	TOTAL CRACKING (m/152.4m)	MAXIMUM RUT DEPTH (mm)	CONE PENETROMETER (Blows/ 300 mm)
1	32.7	38	F	3	573	60	40.2336	3.175	21
2	7.1	6	G	3	370	17	5.1816	11.1125	12
3	4.8	6	P	1	183	37	24.6888	22.225	15
4	32.6	38.1	P	4	448	44	43.2816	6.35	12
5	21.4	19.05	F	4	470	62	37.1856	17.4625	12
6	22.1	19.05	G	5	980	288	72.8472	3.175	42
7	23	19.05	F	5	473	71	65.532	26.9875	21
8	25.6	19.05	F	3	443	37	64.6176	7.9375	15
9	25.4	19.05	P	2	283	21	27.432	19.05	12
10	37.5	38.1	G	3	638	51	10.9728	3.175	25
11	23.6	19.05	G	4	1135	92	11.5824	4.7625	20

TABLE 2 Summary of Voids in Total Mix from In-Place Cores

			VOIDS TOTAL MIX						PCT.
					Outer Wheel Path		Between Wheel Paths		PVMT.
SITE	LAYER	MIX TYPE	AVG (%)	STD (%)	AVG (%)	STD (%)	AVG (%)	STD (%)	> 8% VTM (%)
1	1	HMA	8.7	1.52	7.4	0.61	10.0	0.88	67
1	2	CIR	10.5	3.42	10.0	1.98	11.0	4.68	77
2	1	SEAL	N/A	N/A	N/A	N/A	N/A	N/A	N/A
2	2	CIR	12.6	1.44	11.2	0.63	13.4	1.13	100
3	1	SEAL	N/A	N/A	N/A	N/A	N/A	N/A	N/A
3	2	CIR	13.3	2.27	11.6	1.73	15.1	1.17	100
4	1	HMA	7.5	1.51	7.1	1.55	7.9	1.54	36
4	2	CIR	8.7	1.58	7.2	0.57	10.0	0.52	67
5	1	HMA	3.3	1.75	2.2	0.92	4.2	1.78	0
5	2	CIR	6.9	1.65	6.0	1.15	7.7	1.76	25
6	1	HMA	6.0	2.05	4.6	0.90	7.3	1.98	16
6	2	CIR	8.0	2.17	7.5	0.42	8.5	2.97	50
7	1	HMA	4.9	1.55	3.7	0.67	6.0	1.30	2
7	2	CIR	8.9	2.93	7.5	3.36	10.4	1.81	63
8	1	HMA	7.5	1.56	6.7	1.67	8.3	1.05	37
8	2	CIR	8.1	1.76	7.7	1.93	8.5	1.68	52
9	1	HMA	9.2	1.32	8.3	0.73	10.0	1.19	82
9	2	CIR	8.0	1.28	7.0	0.80	8.7	1.09	48
10	1	HMA	9.0	1.95	7.9	1.20	9.8	2.09	69
10	2	CIR	9.4	1.09	9.0	0.77	9.7	1.26	89
11	1	HMA	7.7	2.03	6.3	1.40	9.5	0.82	44
11	2	CIR	10.1	2.54	9.8	3.41	10.4	2.00	79

N/A = Seal Coat, Not Applicable

testing. At least two cores were used to determine the maximum theoretical specific gravity according to ASTM D2041. The average voids in the total mix (VTM) of the test section and the average VTM for in and between the wheel paths was determined for each layer at each site based on the TMD and bulk specific gravity. The cores were too irregular in shape to determine voids volumetrically. The results of the voids analysis and their correlation with rutting and cracking are shown in Table 2.

Two cores were extracted to determine the asphalt cement content (ASTM D2172) and the gradation of the mineral aggregate (ASTM C117 and C136). The percent passing the No. 4 and No. 200 sieves is shown in Table 3. The extracted aggregate was examined to determine the number of crushed faces of the coarse aggregate (retained on the No. 4 sieve) and the angularity of the fine aggregate (passing No. 4 sieve) determined using the KDOT flow test (*4*), a modification of the National Aggregate Association uncompacted voids test, Method A. Hudson's A (*5*), a measure of the fineness of an aggregate blend similar to the fineness modulus, was calculated from the gradation analysis and the results are shown in Table 3 along with the aggregate angularity measurements. The respective correlations with rutting and cracking are shown in Table 3 as well.

The asphalt cement was recovered from the extracted residue (ASTM D1856), and the absolute and kinematic viscosity (ASTM D2171 and D2170) and penetration (ASTM D5) were determined. The specific gravity of the asphalt cement was not determined, but an assumed value of 1.000, typical for Kansas asphalts, was used in subsequent voids calculations. The remaining 102-mm cores were tested for indirect tensile strength (ASTM D4123) and compressive strength (ASTM D1074). The recovered asphalt cement properties and strength test results, as well as their correlations with rutting and cracking, are shown in Table 4.

Recompacted Mix Properties

The 152-mm cores heated, broken up and combined by layers, and recompacted using the U.S. Army Corps of Engineers CIR Mix Design Method (6) (50-blow Marshall at 135°C), and the gyratory testing machine (GTM) at 620 kPa ram pressure, 1-degree gyration angle, 150 revolutions. Triplicate samples were made at each compactive effort if sufficient material was available. The samples were then tested for unit weight, VTM, voids in mineral aggregate (VMA), voids filled with asphalt (VFA), and Marshall stability and flow. The GTM parameters of gyratory shear index (GSI), gyratory elastoplasticity index (GEPI), and GTM shear strength were also recorded for GTM compacted samples in accordance with ASTM D3387. The results of the recompacted CIR mixtures and their correlations with performance are shown in Table 5.

Analysis of Data

Field Data

The field data were analyzed by overlay type to determine their effect on pavement performance. Transverse cracking was the major distress noted in the initial review of the projects. As demonstrated in Table 1, the seal coats had the least cracking (14.9 m per 152.4-m section), followed by the 38-mm overlays (31.5 m per 152.4 m) and the 19-mm overlays (46.5 m per 152.4 m). The 19-mm HMA overlays were carrying the most traffic—95.2 (ESALs)—followed by the 38-mm overlays (51.7 ESALs) and the seal coats (27.0 ESALs). Site 6, 19-mm overlay, carried three times more ESALs than the next-heaviest traveled site. Excluding Site 6, the 19-mm overlays still carried more traffic (56.6 ESALs) than the 38-mm overlays. The increased traffic loading for the 19-mm overlays could be a significant factor in the higher cracking. The low cracking in the seal coats could be the result of their ability to flow and seal small cracks at the elevated temperatures experienced during the summer in Kansas. The cone penetrometer data showed the subgrades to have similar resistance to penetration with the exception of Site 6. The good performance of Site 6, despite its high traffic, could be related to its better subgrade support as indicated by its higher penetrometer value. The rut depth data shown in Table 1 indicated an increase in rut depth with a decrease in overlay thickness.

Construction Data

Table 2 demonstrates that the average void content of the HMA overlays was 7.1 percent with three of nine sites having an average VTM above 8.0 percent, indicating an air- and water-permeable surface (7). Using the standard deviations of the overlays, the expected percent of each test section with surface voids over 8 percent is also shown in Table 2. Only two sites, Sites 5 and 7, would be expected to have voids below 8 percent for their entire length. Of the three sites with average in-place voids over 8 percent—Sites 1, 9, and 10—the proportion of the test section with voids over 8 percent ranged from 67 to 82 percent. Site 1 was rated fair, Site 9 poor, and Sites 9 and 10 had severe moisture damage.

The Corps of Engineers (6) recommends that CIR be compacted to a maximum of 14 percent voids. All sites

TABLE 3 In-Place Aggregate Properties

SITE	LAYER	HUDSON'S A	PERCENT PASSING No. 4 (%)	PERCENT PASSING No. 200 (%)	COARSE AGGREGATE 2 or MORE CRUSHED FACES (%)	FINE AGGREGATE UNCOMPACTED VOIDS (%)
1	1	5.74	81	10.0	72	38.1
2	1	N/A	N/A	N/A	N/A	N/A
3	1	N/A	N/A	N/A	N/A	N/A
4	1	4.61	55	7.1	74	42.6
5	1	5.75	82	9.0	77	37.1
6	1	5.56	77	10.6	61	37.7
7	1	5.60	77	11.7	23	40.0
8	1	5.35	75	8.2	86	39.7
9	1	5.52	83	7.6	84	39.1
10	1	5.42	75	7.7	86	39.8
11	1	5.49	79	8.5	68	38.4
1	2	5.91	86	13.3	61	39.8
2	2	5.82	84	13.8	72	38.8
3	2	6.33	91	16.0	63	39.2
4	2	5.74	80	12.1	74	38.7
5	2	5.93	82	13.3	86	37.4
6	2	5.76	79	11.1	40	39.7
7	2	6.27	89	14.9	8	40.3
8	2	5.77	79	12.8	48	39.6
9	2	5.80	86	11.3	43	38.9
10	2	6.03	89	13.7	63	39.3
11	2	5.83	85	11.1	62	38.9
Rutting R		0.65	0.48	0.59	-0.37	-0.04
Alpha*		0.03	0.14	0.07	0.26	0.90
Cracking R		-0.05	-0.46	-0.11	-0.67	0.44
Alpha*		0.88	0.15	0.75	0.07	0.17

* (1-Alpha) = Probability R Not Equal to 0.
N/A = Seal Coat, Not Applicable.

TABLE 4 Summary of In-place Strength Parameters and Recovered Asphalt Properties

SITE	LAYER	INDIRECT TENSILE STRENGTH (kPa)	INDIRECT TENSILE STRAIN (mm/mm)	COMPRESSIVE STRENGTH (kPa)	H/D RATIO	ASPHALT CONTENT (%)	PEN 25 C	RECOVERED VISCOSITY ABSOLUTE (poise)	RECOVERED VISCOSITY KINEMATIC (cSt)
1	2	431	0.15	3897	0.72	4.80	25	8263	1397
2	2	572	0.15	3996	0.53	4.91	27	12112	1071
3	2	260	0.17	1164	0.65	4.88	39	12114	683
4	2	512	0.14	2119	0.75	5.25	N/T	13919	858
5	2	469	0.19	2075	0.89	5.48	43	141594	642
6	2	508	0.15	3024	0.99	5.48	29	15079	1578
7	2	352	0.27	N/T	N/T	5.37	44	24049	661
8	2	562	0.16	3096	0.56	4.71	24	26786	1630
9	2	401	0.23	N/T	N/T	5.86	45	37625	738
10	2	386	0.23	2986	0.75	5.32	N/T	71889	1132
11	2	570	0.16	3145	0.87	4.63	N/T	82630	935
Correlation Analysis									
Rutting R		-0.60	0.61	-0.68	N/A	0.34	0.87	-0.49	-0.73
Alpha*		0.05	0.05	0.04	N/A	0.31	0.01	0.13	0.01
Cracking R		0.01	0.04	-0.09	N/A	0.20	-0.11	0.15	0.36
Alpha*		0.97	0.91	0.82	N/A	0.55	0.80	0.67	0.27

N/T = Not Enough Material to Test.
N/A Not Applicable.
* (1-Alpha) = Probability R Not Equal to 0.

TABLE 5 Summary of Recompaction Analysis

SITE	LAYER	UNIT WEIGHT (kN/m^3)	VTM (%)	VMA (%)	VFA (%)	MARSHALL STABILITY (kN)	MARSHALL FLOW	GSI	GEPI	SHEAR STR. (kPa)
GTM RECOMPACTED, at 620 kPa, 1 DEGREE, 150 REV., 135 C										
1	2	23.3	2.1	13.4	84.6	20.0	17.2	1.36	1.52	56.0
2	2	23.3	1.5	13.0	88.8	12.9	20.0	1.48	1.62	34.0
3	2	23.1	1.8	13.2	86.2	14.8	20.3	1.41	1.61	46.0
4	2	23.3	1.0	12.7	92.3	16.2	18.3	1.60	1.52	43.0
5	2	23.3	0.3	13.3	97.4	13.0	19.3	1.63	1.57	18.0
6	2	23.2	1.0	13.8	93.1	16.2	16.0	1.44	1.52	30.0
7	2	23.2	0.9	13.8	93.2	10.4	23.3	1.60	1.58	0.0
8	2	23.4	0.7	11.8	93.8	14.9	19.0	1.59	1.57	15.0
9	2	23.1	0.5	15.2	96.7	12.4	18.3	1.56	1.60	73.0
10	2	22.7	1.3	13.5	90.5	18.6	14.0	1.37	1.48	113.0
11	2	23.3	1.9	12.8	85.3	22.1	15.3	1.14	1.52	154.0
Correlation Analysis										
Rutting R		-0.04	-0.35	0.36	0.33	-0.79	0.83	0.48	0.75	-0.45
Alpha*		0.91	0.30	0.27	0.31	0.00	0.00	0.13	0.01	0.16
Cracking		0.36	-0.43	-0.06	0.42	-0.30	0.31	0.49	-0.07	-0.66
Alpha*		0.28	0.19	0.85	0.19	0.37	0.36	0.12	0.83	0.03
MARSHALL RECOMPACTED, AT 135 C										
1	2	23.1	2.9	14.6	85.0	20.1	13.5	N/A	N/A	N/A
2	2	23.1	2.1	13.5	84.6	15.1	15.7	N/A	N/A	N/A
3	2	23.0	2.5	13.9	81.7	15.8	13.5	N/A	N/A	N/A
4	2	23.2	1.7	13.3	87.1	14.5	14.2	N/A	N/A	N/A
5	2	23.1	1.3	14.1	90.6	11.4	15.8	N/A	N/A	N/A
6	2	22.9	2.2	14.9	85.1	14.5	13.8	N/A	N/A	N/A
7	2	23.0	1.5	14.3	89.4	11.5	17.5	N/A	N/A	N/A
8	2	23.1	2.0	13.0	84.7	13.7	14.8	N/A	N/A	N/A
9	2	22.9	1.4	14.9	90.8	11.0	14.5	N/A	N/A	N/A
10	2	22.6	1.8	14.0	86.8	18.5	13.7	N/A	N/A	N/A
11	2	23.1	2.7	13.5	79.9	16.0	12.8	N/A	N/A	N/A
Correlation Analysis										
Rutting R		0.06	-0.48	0.17	0.41	-0.67	0.65	N/A	N/A	N/A
Alpha*		0.86	0.14	0.62	0.22	0.02	0.03	N/A	N/A	N/A
Cracking		0.20	-0.18	0.21	0.27	-0.35	0.32	N/A	N/A	N/A
Alpha*		0.56	0.60	0.53	0.42	0.30	0.33	N/A	N/A	N/A

N/A = Not Applicable.
* (1-Alpha) = Probability R Not Equal to 0.

had CIR voids below 14 percent, indicating adequate compaction. Sites 2 and 3 had the highest voids of the CIR mixes and both had seal coats. The seal coat at Site 2 was intact and Site 2 was rated as good, whereas Site 3 had a failed seal coat and the poorest performing pavement. The presence or lack of moisture in CIR appears to have a pronounced effect on performance. The literature (*1,8,9*) indicates that CIR is susceptible to moisture damage. The data in Table 2 indicate that the surface mixes are not performing one of their primary functions, that is, providing a waterproof barrier to the remainder of the pavement. The high in-place voids of the HMA overlays could indicate that the CIR does not give adequate support for compaction of thin overlays.

Aggregate Properties

The results of the gradation analyses (Table 3) show that the CIR mixes average 15.5 percent coarse aggregate, with a range of 9 to 21 percent; 71.5 percent fine aggregate, with a range of 66 to 75 percent; and 13 percent filler, with a range of 11 to 16 percent. In all instances the filler content is above the maximum amount of 10 percent allowed by current KDOT specifications (*10*). By comparing the percent passing the No. 200 sieve for the Layer 1 and CIR mixes, it appears that the milling operations increase the fines content of the CIR by several percent.

The CIR mixtures were very fine, as indicated by Hudson's A. The average Hudson's A was 5.93, whereas a typical KDOT 12.7- and 19-mm dense graded surface mix (*10*) would have a Hudson's A of 5.41 and 4.67, respectively. Figure 1 shows the relationship between Hudson's A and rutting. The relationship has an R^2 of 0.44 and shows that the finer the gradation (higher Hudson's A), the greater the rut depth. The correlation coefficients for gradation also indicate that as the percent passing the No. 4 and No.

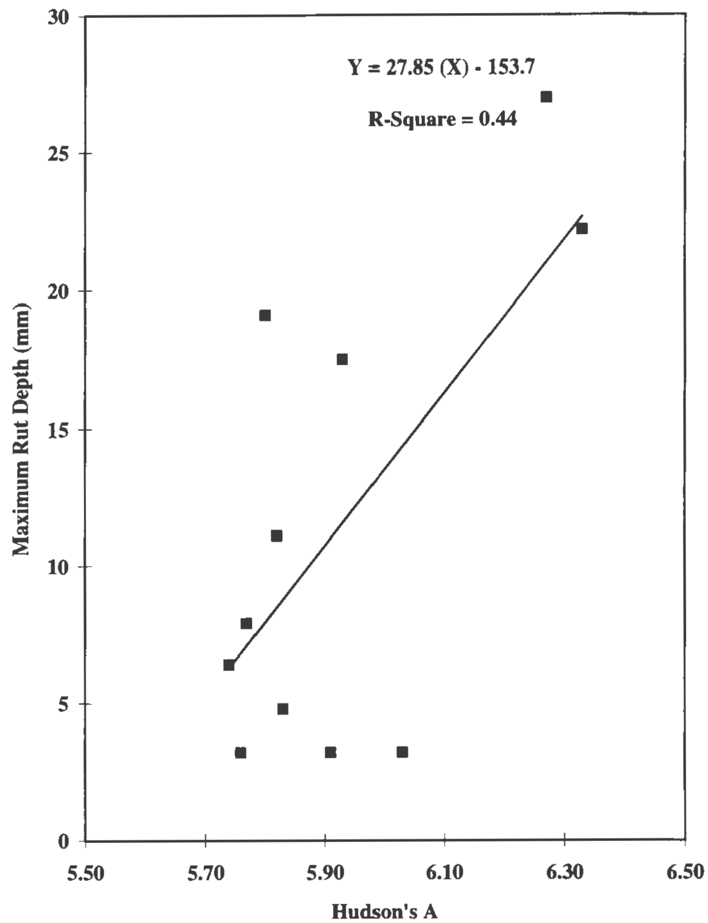


FIGURE 1 Hudson's A versus maximum rut depth.

200 sieves increases, indicating a finer mix, the rut depth increases. The test data illustrate that the locally available aggregates are rounded and poorly graded, and that the milling, screening, and crushing operations increase the fines, lowering the quality of the aggregates. The low quality of the aggregates has an adverse effect on the performance of the CIR mixtures.

The fine aggregate in the CIR consisted mainly of rounded natural sands. The uncompacted void contents determined by the KDOT flow test for the fine aggregate ranged from a low of 37.4 percent to a high of 40.3 percent. Based on the author's previous work with Kansas materials (*4*), this would correspond to a mixture of 52 to 97 percent rounded natural sand with an average of 71.5 percent natural sand. The natural sand content of the mixes is well above FHWA recommendations of 20 to 25 percent for low-trafficked pavements (*11*) and is an indication of the scarcity of angular aggregates in western Kansas.

The coarse aggregate typically made up only 15 percent of the CIR and was primarily uncrushed gravel. Table 3 demonstrates that the percent of two or more crushed faces of the coarse aggregate ranged from a low of 8 percent to a high of 86 percent. Figure 2 shows the results of the angularity of the coarse aggregate on cracking. The relationship show that as the percent of two or more crushed faces increases, the incidence of reflective cracking decreases. The relationship is not that strong with an R^2 of only 0.33.

The results of the aggregate testing show that the quality of the locally available material used, as measured by angularity, surface texture and gradation, was low. The correlation analysis, demonstrated in Table 3, indicates that as the quality of the aggregates decreases, as measured by the above properties, the amount of rutting and cracking increases. The importance of aggregate properties in rutting resistance of CIR is shown in Figure 3. The angularity of the aggregate, as measured by the uncompacted void content and crushed face count, and the gradation as measured by Hudson's A predicted the observed rutting with an R^2 of 0.81. There are only 11 data points. However, the relationship illustrates the importance of the gradation and rough-textured aggregates in reducing rutting.

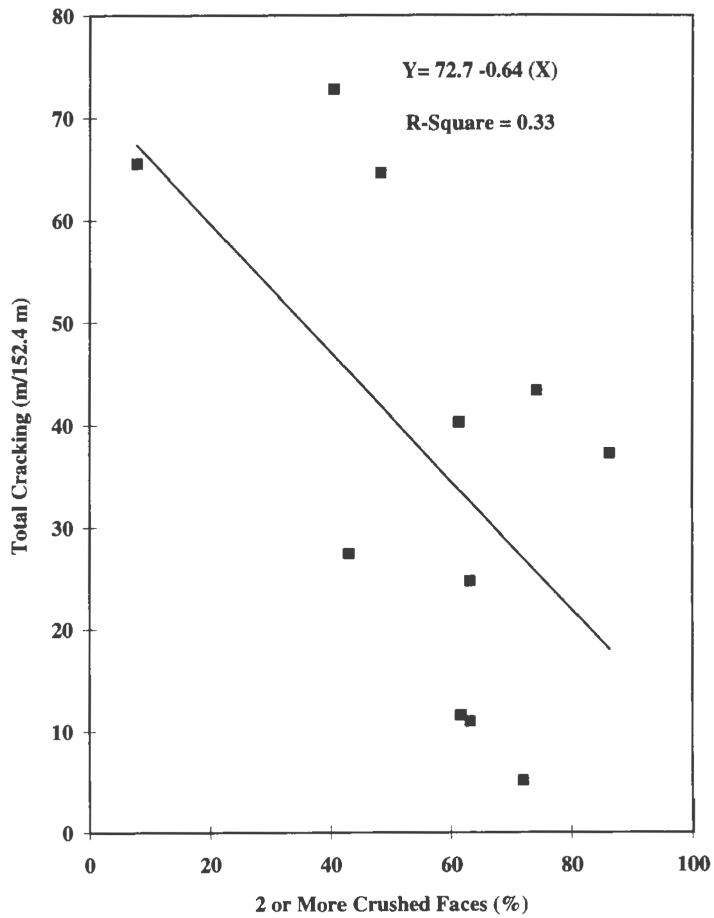


FIGURE 2 Percent two or more crushed faces (CF) versus total cracking.

Mixture-Strength Parameters

The mixture-strength parameters of indirect tensile strength and strain at failure and compressive strength are demonstrated in Table 4, along with their correlation with rutting and cracking. The compressive-strength results are uncorrected due to the inability to meet the minimum height-to-diameter ratio requirement. The correlation coefficients show significant correlations with rutting but not with cracking. The strength tests were performed on aged samples and the relevance to design situations is doubtful. The correlation coefficients indicate that the higher the strength the better the resistance to rutting. The reported strengths are low, especially when compared to conventional hot-mix asphalt, and these low strengths could be a function of the high in-place voids, excess fines and low angularity of the CIR mixes.

Recovered Asphalt Cement Properties

The results of the recovered asphalt cement properties and their correlations with rutting and cracking are shown in Table 4. The recovered asphalt cement properties of viscosity and penetration had some of the better correlations with rutting and cracking. However, during the Abson's recovery process the new and old asphalts are mixed. It is doubtful that this happens to the same extent with the low temperatures associated with CIR mixes. The interpretation of the results is difficult for this reason, and conclusions drawn would be suspect. Therefore, the results are presented for information only.

Recompacted Mix Properties

The recompaction analysis of the CIR mixes on both the GTM with 50 blows per side with the Marshall

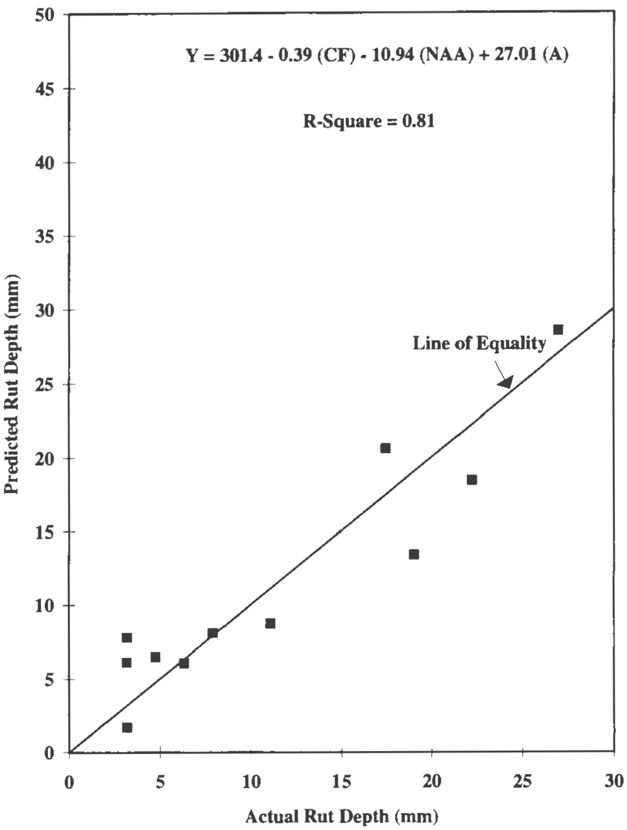


FIGURE 3 Two or more crushed faces (CF), uncompacted void content (NAA), and Hudson's A (A) versus maximum rut depth.

hammer at 135°C (Corps of Engineers method) and their correlation with rutting and cracking are shown in Table 5. The best relationships for the Corps of Engineers method were found between rutting and Marshall stability and flow. The correlation coefficients indicate that as the Marshall stability increases and flow decreases, the rut depth decreases. The tests were performed on aged samples, and the effect of aging on Marshall stability is well documented. Therefore, Marshall stability from recompacted samples should not be used for design parameters. None of the 11 CIR mixes met the mix design requirements of the Corps of Engineers procedure (7) of minimum 76-kN Marshall stability, maximum flow of 20, 3 to 5 percent VTM, and 75 to 85 percent VFA. All of the 11 mixes had recompacted voids below the minimum range of 3 percent, and only 5 mixes met the VFA requirement. Since none of the recompacted samples met the minimum Corps of Engineers method requirements, the effectiveness of the method was difficult to evaluate. However, the Corps of Engineers method did indicate the need for additional aggregate to improve gradation to meet the design requirements.

The GTM recompacted samples had higher correlation coefficients than the Corps of Engineers method. The Marshall stability and flow were correlated with rutting. However, their usefulness is limited for the reasons given above. The GEPI, a measure of the internal friction of the aggregate blend, was correlated with rutting. The relationship is shown in Figure 4 and has an R^2 of 0.56. As the GEPI increases (indicating a decreasing internal friction angle) the rutting increases. The internal friction of the aggregate is related to the angularity and surface texture and size of the aggregate and indicates that more angular aggregates are needed to resist rutting.

The relationship between GTM shear strength and cracking is shown in Figure 5. The relationship has an R^2 of 0.44 and shows that as the GTM shear strength of the mix increases the total cracking decreases. The majority of the cracking encountered was reflective cracking and the relationship indicates that enhanced shear strength appears to impede the reflection of the existing crack. The GTM shear strength, shown in Figure 5, predicted cracking with as high a degree of confidence as any other single parameter evaluated. The GTM shear strengths are low and the GSI and GEPI values are high when compared to typical dense-graded hot mixes, indicating the low strength and angularity of the locally available materials. From the above analysis it is clear that the angularity, gradation and shear strength of the CIR have a pronounced effect on the performance of CIR.

Rutting and Cracking Models

The GTM parameters of GSI, GEPI and shear strength would be available when the mixture was being designed. Using the GTM properties and the aggregate properties, a model was developed to predict rutting. The model has an R^2 of 0.92 and is shown in Figure 6. The model shows the importance of the angularity and gradation of the aggregate. Milling changes the gradation of the mixture. Therefore, millings rather than cores should be used to evaluate aggregate properties.

To evaluate the cracking potential of CIR, the shear strength and traffic in total ESALs predicted cracking with an R^2 of 0.64. The relationship is shown in Figure 7. The relationship indicates that the shear strength, which can be improved by improving aggregate properties and compaction, as well as traffic affect performance. The relationship indicates the importance of thickness design, even for maintenance treatments.

Conclusions and Recommendations

The analysis of the data indicates that the performance of CIR in Kansas is related to the poor angularity and

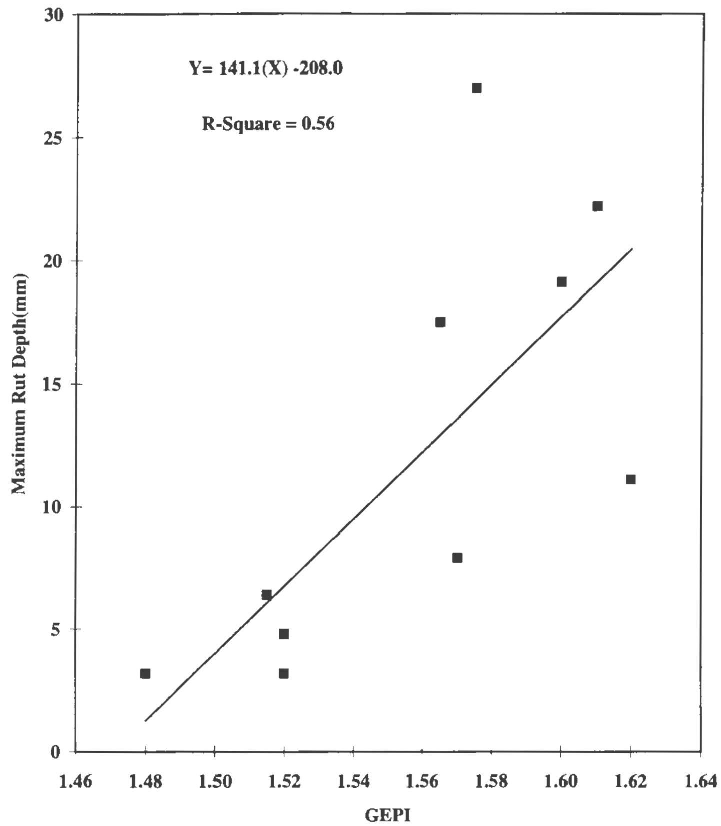


FIGURE 4 GEPI versus maximum rut depth.

size/gradation of the locally available aggregates. The aggregates are rounded, poorly graded, and have excess fines when compared to traditional dense-graded mixes. When recompacted on the GTM, the CIR mixtures were unstable as indicated by high GSI and low VTM. The CIR mixtures had low shear strengths and the GEPI indicated low angels of internal friction. Poor compaction of the HMA overlays, as indicated by high in-place voids, contributed to the poor CIR performance. The high voids in the HMA surface mix allowed water to enter the CIR and damage the mix. The relationship between shear strength, traffic and cracking indicate the importance of mix strength and adequate thickness design, even for maintenance treatments.

Based on the results of this study the following recommendations were made to improve the performance of CIR:

1. Improve compaction of the HMA overlays to prevent water from entering the CIR and damaging the mix.
2. Use millings as opposed to cores to design and evaluate CIR mixtures.
3. Use the relationship in Figure 3 to evaluate aggregate properties and rutting potential of CIR mixtures.
4. Compact samples on the GTM as previously described and evaluate the rutting potential using the surface area, recompacted voids and GEPI as shown in Figure 6 and the cracking potential using the shear strength of the mix and the total anticipated traffic as shown in Figure 7.
5. If the predicted rut depths or cracking is unacceptable, investigate aggregate benefaction or chemical stabilization.

Based on the results of this study, the two options for improving CIR performance were aggregate benefaction or chemical stabilization. Adding crushed coarse aggregate to the CIR should improve the performance of the CIR. However, high minus No. 200 content could require 25 to 50 percent coarse aggregate to reduce the minus No. 200 material to an acceptable limit. Adding coarse aggregate would do little to improve the angularity of the fine aggregate which affects rutting (Figure 3). The high percentage of coarse aggregate required could cause constructability problems and limit the depth of the existing material that could be removed. Due to the expense of importing quality aggregates and the desire to retain the cost effectiveness of CIR, this option has not been adopted by KDOT.

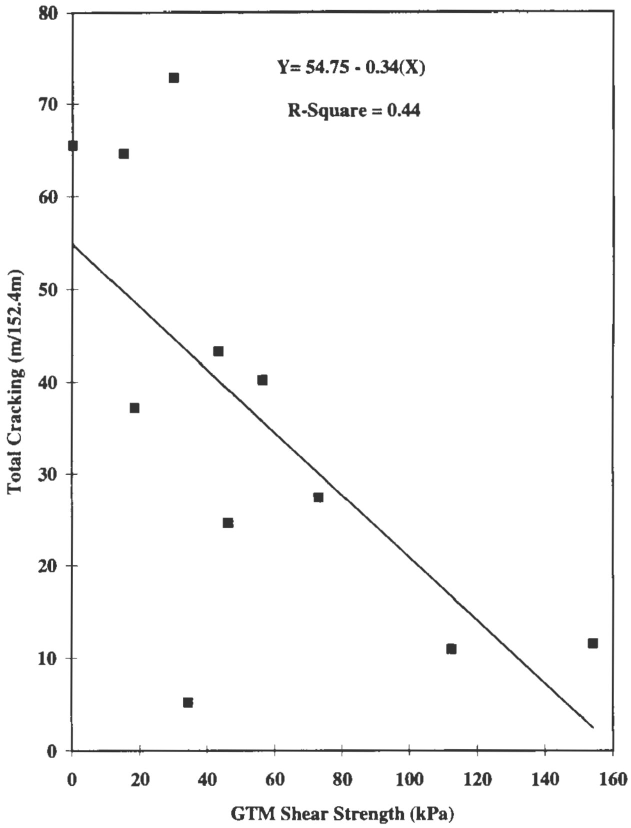


FIGURE 5 GTM shear strength versus total cracking.

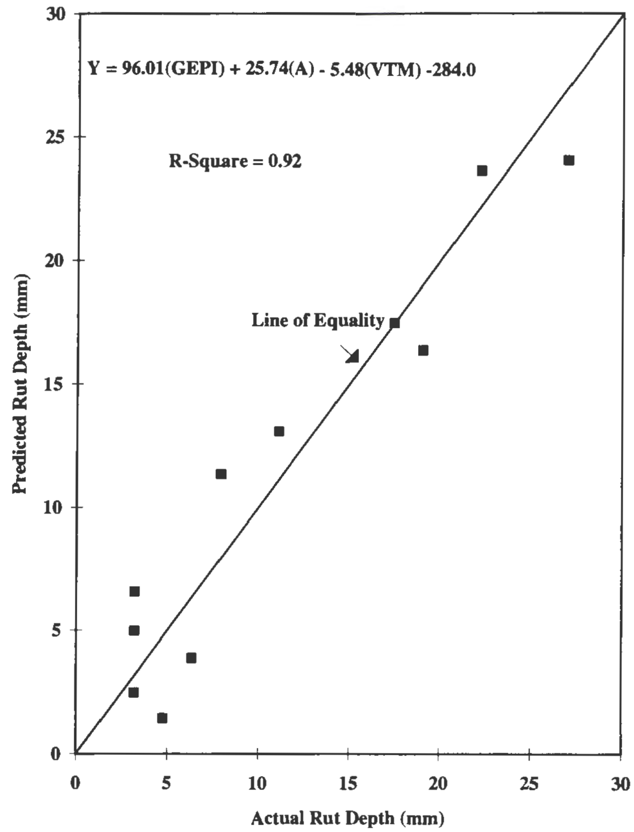


FIGURE 6 GEPI, Hudson's A (A), and GTM recompacted VTM versus maximum rut depth.

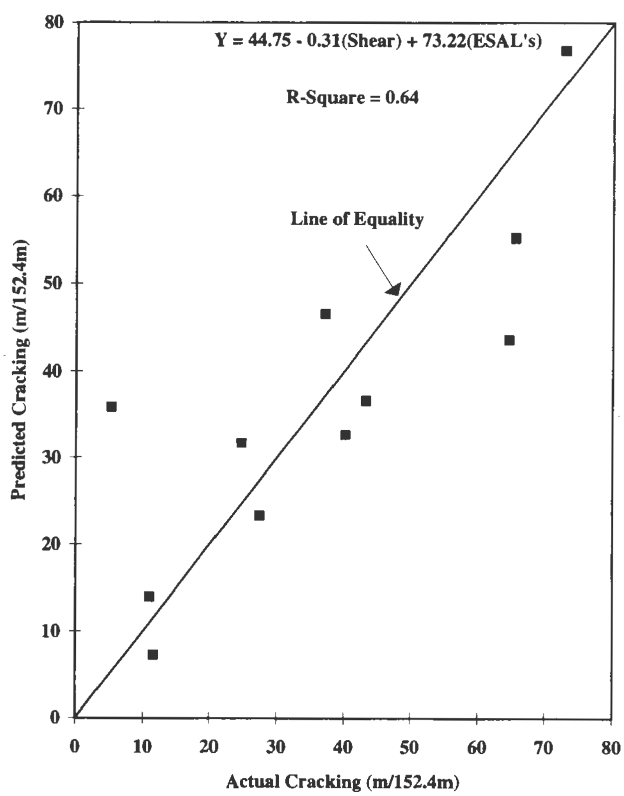


FIGURE 7 GTM shear strength and total traffic (million ESALs) versus total cracking.

The second option, chemical stabilization, is currently being evaluated by KDOT to improve the performance and retain the economics of CIR. Preliminary results from field trials, previously reported by the author (*12*), indicate that Type C fly ash increases the resistance of the mix to moisture damage and increases the strength of the mix which decreases the potential for wheel-path rutting. Chemical stabilization with class C fly ash was the only additive used in the 1994 CIR program.

ACKNOWLEDGMENTS

This project was funded as part of the K-Tran program, a cooperative research program between KDOT and the University of Kansas. The authors are grateful for their support.

REFERENCES

1. Epps, J. A. *Cold-Recycled Bituminous Concrete Using Bituminous Materials, NCHRP Synthesis of Highway Practice 160.* NCHRP Synthesis of Hwy Practice 160: Cold Recycled Bituminous Concrete, National Research Council, Washington, D.C., 1990.
2. Cross, S. A. *Evaluation of Cold In-Place Recycling.* K-Tran Project No. KU-93-1. University of Kansas Center for Research, Inc., Aug. 1994.
3. *1992 Kansas NOS Condition Survey Report.* Bureau of Materials and Research, Kansas Department of Transportation, Topeka, Aug. 3, 1992.
4. Cross, S. A., B. J. Smith, and K. A. Clowers. Evaluation of Fine Aggregate Angularity Using the NAA Flow Test. In *Transportation Research Record 1437*, National Research Council, Washington, D.C., 1994.
5. *NCHRP Report 69: Evaluation of Construction Control Procedures.* TRB, National Research Council, Washington, D.C., 1968.
6. *Guide Specifications for Military Construction, Section 02564, Cold Mix Recycling.* Department of the Army, U.S. Army Corps of Engineers, CEGS-02564, March 1989.
7. Zube, E., Compaction Studies of Asphalt Concrete Pavements as Related to Water Permeability Test. Presented at 41st Annual Meeting of the Highway Research Board, Washington, D.C., 1962.
8. Hanson, D. I., and R. D. Williams. In-Situ Cold Recycling. In *Proceedings, the 25th Annual Paving and Transportation Conference*, University of New Mexico, Albuquerque, New Mexico, 1986.
9. Scholz, T. V., R. G. Hicks, D. F. Rogge, and D. Allen. Use of Cold In-Place Recycling on Low-Volume Roads. In *Transportation Research Record 1291, Vol. 2,* National Research Council, Washington, D.C., 1991.
10. *Standard Specifications for State Road and Bridge Construction.* Kansas Department of Transportation, Topeka, 1990.
11. *Asphalt Concrete Mix Design and Field Control.* Technical Advisory T-5040.27. Federal Highway Administration, U.S. Department of Transportation, March 10, 1988.
12. Cross, S. A., and G. A. Fager. Fly Ash in Cold Recycled Bituminous Pavements. Presented at the 73rd Annual Meeting of the Transportation Research Board, Washington, D.C., January 9–13, 1994.

The opinions expressed in this paper are those of the authors and not necessarily those of KDOT.

Prolonging Haul Over Frozen Roads

Joe Barcomb, *USDA Forest Service, Libby, Montana*

A 5-cm-deep blanket of sawdust was used successfully on a timber haul road in the Kootenai National Forest in northwestern Montana to extend the period of timber hauling from mid-February to the end of March. The annual cost for applying, maintaining, and removing the sawdust from the road was far less than that of rebuilding the road after failure or reconstructing it for all-season haul.

The Kootenai National Forest in northwest Montana has been considering ways to extend timber haul during the spring breakup period from mid-February through March. As the hauling period becomes shorter because of considerations under the Threatened and Endangered Species Act, extending the haul period has become essential to the timely removal of forest products. The USDA Forest Service has used an age-old technique for insulating roads while at the same time has begun to perform some state of the art nondestructive testing to analyze how it may modify or lift road use restrictions.

The project road is located in northwest Montana near the Canada-U.S. border, with a rise of approximately 850 m in about 14 km. Much of the timber haul is limited to the 4-month period between December and March because of restrictions placed for the protection of grizzly bears. This haul season is further shortened because of spring breakup, thawing of the road bed. One lumber company suggested that hauling might be extended beyond the normal spring breakup time by applying sawdust to the snow-covered road surface, thereby insulating the road prism.

The Kootenai National Forest gave approval for field-testing of the sawdust/insulation proposal in January 1993. The mill provided and placed sawdust on the lower-elevation segment of the road and was able to extend the period of frozen road haul into late March.

Description of Road

The project road, Kootenai Road Number 92, is located about 15 km south of the Canada-U.S. international border and 50 km east of the Idaho-Montana state border (see Figure 1). By road, the city of Eureka, Montana, lies approximately 30 km east of the beginning of the project.

Forest Service Road Number 92 was completed in 1984 with approximately 7.5 cm of asphalt cement pavement and 15 cm of crushed aggregate base over the subgrade. The subgrade is a glacial till that has a high silt content and is therefore very susceptible to frost action. The high spring groundwater and relatively slow-draining silt keep the subgrade saturated until late in the spring. The road does not regain most of its strength for about 10 weeks.

The mountainous project road starts at an elevation of about 875 m at the junction of a Lincoln County road and twists and turns as it climbs on an average of over 6 percent for 13.8 km to the pass, where the elevation is 1715 m.

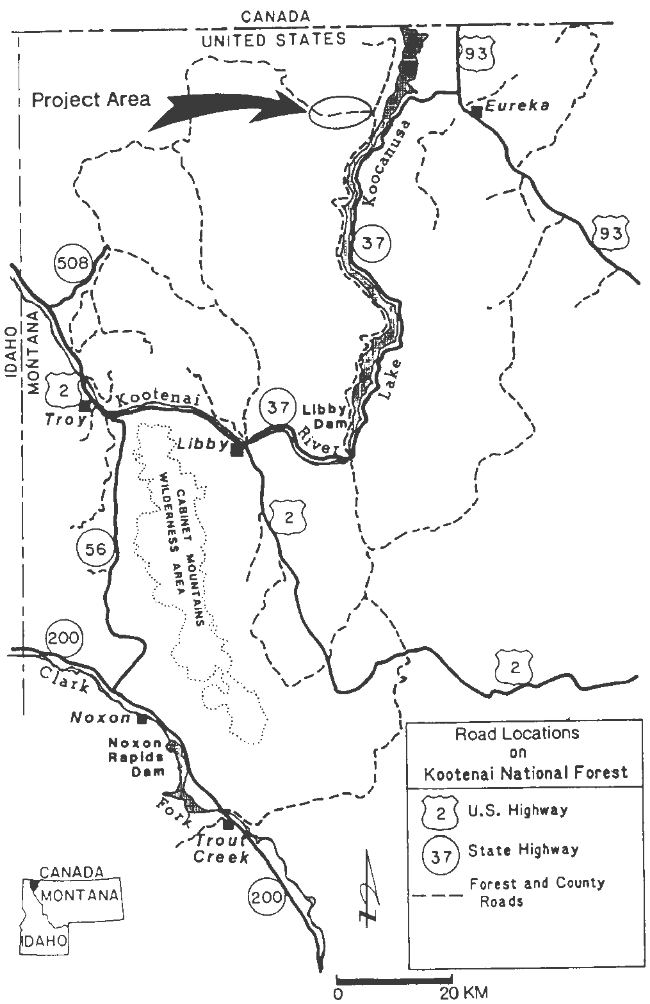


FIGURE 1 Location of project area.

Road Management

The Forest Service is in a unique position to manage its road system. Since technically it is not a public road agency, it can easily restrict size, weight, speed, and type of vehicle. With the ability to close roads or severely restrict their use, the Forest Service very seldom designs roads to support traffic loads for all-season use because of the additional cost of providing for the few weeks of worst subgrade conditions. The same economics affect most low-volume roads elsewhere in the world; they also are not built to provide all-season full traffic load support.

The Kootenai National Forest uses data collected from thermistor strings (*1*) installed in many of its roads to restrict road use during spring breakup. The thermistor string consists of a multistrand electrical cable with several thermistor sensors, small semiconductors whose electrical resistance varies with temperature fluctuations that are attached and sealed at specified intervals. Data from these thermistor strings show that there can be a 2- to 3- month delay from the time the road goes into spring breakup at the lower elevation until it reaches breakup conditions at the summit.

Dynamics of Breakup

Frost penetration on this road often reaches a depth of 1.5 m. Latent internal heat prevents the frost from much greater penetration. It also thaws the lower frost depths when surface temperatures rise. The rate of thaw from below is much slower than the rate caused by rising surface temperatures caused by the sum on the asphalt pavement.

The sun does not reach the pavement until the ice-and-snow cap, up to 30 cm thick, has melted. There is a general requirement not to bare road surfaces during plowing operations.

Because of the twisting-turning orientation of the road and the various shading conditions, the ice-and-snow cap melts at various rates within the same general elevation zone. Some sections will melt a month sooner than others, which means that some sections of the road will be completely frost-free before thawing begins on other sections.

Problem

When the road was built, logging activities were allowed during most of the year, except during the spring breakup when the roads lost their strength and the ground became too saturated for yarding activities. Since then a grizzly bear recovery plan has been implemented, which affects much of the logging activity in the Kootenai National Forest.

The restrictions have been placed to improve habitat conditions for the grizzly bear, an endangered species. In many areas of the forest, timber harvest is now allowed only when the bears are hibernating in their dens, from late November to late March. Much of the timber harvesting is contractually restricted to a 4-month period: December, January, February, and March.

General timber truck haul does not occur until sufficient quantities of timber have been cut and brought to landings beside local roads. Once trucking is started, the operators want sufficient logs beside the roads so that haul can continue uninterrupted. It takes 2 or 3 weeks to ensure that there is sufficient timber beside the roads. This operation takes place during the first 2 to 3 weeks in December, before the general shutdown for Christmas and New Year.

Timber haul normally starts shortly after the first of the new year. Spring breakup usually occurs around mid-February at the lower elevations, and the Forest Service imposes load limit road restrictions. The hauler who used to be able to operate trucks for 5 months or more is now limited to the 6 weeks between January 1 and mid-February.

Solutions

The timber purchaser and the Forest Service started looking for means to extend the haul season through March. The obvious solution was to build an all-season haul road, but an estimated cost of $150,000 to $250,000 per kilometer dampened enthusiasm.

It was then decided that the entire road would not have to be rebuilt, since only the first 3 or 4 km at the lower elevation actually went into spring breakup before the end of March. Even the cost for rebuilding only 3 to 4 km seemed staggering: $500,000 to $1,000,000.

Another solution considered was to allow haul on the road during spring breakup; that solution was rejected because it would have caused the pavement to be destroyed every 2 to 3 years. Replacement costs are estimated at $50,000/km. Again, a cost of $50,000 to $75,000 per year to replace the lower 3 km seemed too high.

One of the company owners suggested that sawdust be placed on the road in late winter. The sawdust would act as an insulation and prevent the thaw of the ice-and-snow cap on the road surface, therefore delaying the thawing of the road structure.

There is good historical evidence of the insulating value of sawdust. It was used extensively to protect ice in ice-houses until modern refrigeration replaced the use of ice for food preservation. Sawdust can still be found within some old building walls. It was used before the advent of modern insulation.

The company requested permission to apply 10 cm of sawdust to the road in January 1993. The Forest Service agreed to the proposal, but continued to make arrangements for nondestructive testing.

Field Trial and Results of Sawdust Application

In February 1993 the purchaser hauled 560 m^3 of sawdust and spread it on those sections at the lower elevations that were the most exposed to the sun. The hauling vehicle, a truck-tractor with an 80-m^3 self-unloading trailer, made the primary laydown and was followed by a grader that completed the spread.

Sawdust was placed on road segments for a total of a little less than 1 km. The sawdust was laid down approximately 6 m wide and 10 cm deep.

Initially the logging trucks had trouble climbing the grade with 10 cm of loose sawdust on the ice-and-snow pack. Five cm of sawdust was bladed off the road after the second day of timber haul. Thereafter, when the sawdust was incorporated into the ice-and-snow layer, the truckers reported better traction.

Trucks continued to haul timber over the road until the end of March. At that time areas above the segments receiving sawdust were starting to develop exposed pavement and were going into spring breakup. The surface was bladed once during the haul period to remove ruts that had developed in the ice-and-snow and sawdust. Late in April the sawdust was bladed off the road onto the shoulders and fills in an attempt to suppress weed growth. The remnants left on the road after blading were allowed to blow off by the winds generated by traffic.

The field trial proved to be so successful that spot applications of sawdust 5 cm deep were placed on the road again in February 1994.

Costs

The 1993 costs were less than $3,000, which included the sawdust, hauling, spreading, maintaining, and removing. The sawdust was valued at \$1.40/m^3. This is the approximate price that the mill can receive for the sawdust when it is used as fuel.

In 1994 the total cost is expected to be about $1,500 even though the sawdust will be picked up and hauled to storage. The cost will be lower because less sawdust will be used—5 cm instead of 10—and haul to the project will be reduced from about 30 km to about 12, as sawdust is coming from a closer mill.

Environmental Concerns

The sawdust alters the immediate roadside environment. The effects of blading the sawdust to the side of the road were both positive and negative: it reduced noxious weeds on the shoulder and upper part of the fills, but at the same time it also killed desired grass species. The total amount of biomass that was introduced to the landscape is no greater than that found naturally on much of the forest floor on either side of the road.

The biggest concern is the potential fire hazard. Normally the roadsides have much less fuel on the shoulders and cut-and-fill slopes. It is feared that the careless presence of travelers on the road could lead to a potential fire in the sawdust along the road shoulder, which could spread to the surrounding forest.

There is little concern about the impact of the sawdust on the natural view, since most of the traffic is related to timber harvest. The people involved in timber harvest see the sawdust as a solution and not a deterrent.

Forest insects and diseases are not a major concern since the sawdust is being developed from locally harvested timber. Concern would be much higher if the sawdust were coming from milled timber that was not from the immediate area. Two other factors in normal milling activities reduce the potential of spreading diseases and insects. Most mills debark the logs before sawing. A large percentage of insects live just under the bark in the cambium and are destroyed or removed with the bark.

In addition, the sawdust used on this project had been placed in large piles, which through natural decomposition develop high internal heat that would kill most forest diseases.

There are no large bodies of surface water crossing or adjacent to the road. Most surface flow is caused by snow melt. Almost all the sawdust moved by snow melt or heavy rains would be trapped in the natural vegetation and debris on the forest floor below the road. Insignificant amounts might find their way into a channel.

Future operations will remove most of the sawdust from the road and place it in easily accessible stockpiles so it can be used repeatedly in future years. The small amount left on the road will be dispersed into the surrounding forests by mechanical brooming or by the traffic-generated wind.

Past Uses

Shortly after the first trial in 1993, a retired woodsworker came forward with the information that the same process had been used on some gravel surface roads in the early 1940s to extend timber haul in the spring. It was equally successful.

Applications for Sawdust Use by Others

Sawdust might be used by other agencies or industries to extend the period of haul in the spring when the following conditions exist:

1. Frost depths under roads reach or exceed 1 m,
2. Ten to Fifteen cm of compacted snow or ice can be tolerated on the road surface,
3. Traffic speeds are generally less than 40 km/hr, and
4. Traffic volumes do not support the cost of providing an all-weather road.

Sawdust was the material of choice on this project because of its availability and relative inexpensiveness. Another material with good insulating qualities and durability could easily be substituted. What was done on this project for the short term is not different from what has been accomplished for the long-term insulation on roads built in permafrost.

Summary

Sawdust has been successfully used in northwestern Montana to insulate a road in spring. The insulating effect delayed thawing of the road structure thus allowing timber haul to be extended by weeks. The annual cost for applying, maintaining, and removing the sawdust from the road is far less than that for rebuilding the road after failure or reconstructing it for all-season haul.

REFERENCE

1. McBane, J., and G. Hanek. Determination of Critical Thaw-Weakened Periods in Asphalt Pavement Structures. In *Transportation Research Record 1089*, TRB, National Research Council, Washington, D.C., 1986, pp. 138–146.

STABILIZATION

Performance Evaluation of Tall Oil Pitch Emulsion for Stabilizing Unpaved Forest Road Surfaces

D.M. Bennett, *Forest Engineering Research Institute of Canada*
K. Gleeson, *British Columbia Ministry of Forests, Canada*

In British Columbia's central interior, production of tall oil-based road-surface stabilizers, which are coproducts of the kraft pulping process, began in 1990. One product, a tall oil pitch (TOP) emulsion, was applied as a test to a 5.1-km section of unpaved forest road near Prince George, British Columbia, in May 1992. The Forest Engineering Research Institute of Canada cooperated with the British Columbia Ministry of Forests' Prince George Forest District to evaluate the emulsion's performance on the forest road. Road-surface conditions and dustfall were monitored, and information about application procedures and costs, aggregate gradation, climate and traffic conditions, and maintenance procedures was collected.

Maintenance of a high-quality running surface on unpaved forest roads is an ongoing challenge for forest companies and government agencies. A stabilized surface made up of correctly graded aggregate can lower road and vehicle maintenance costs, lengthen the interval between resurfacing treatments, improve road safety and public goodwill by reducing dustfall, and reduce sediment in runoff water. However, because it is difficult to quantify these benefits monetarily, forest companies are often reluctant to invest in road-surface stabilization projects.

The forest industry in British Columbia has used calcium and magnesium chlorides, calcium and sodium lignosulfonates, asphalt emulsions, seal coats, and other products to stabilize unpaved road surfaces and control dust. Recently several lesser-known compounds, some of them proprietary, have emerged in North America as alternative treatments. They are referred to as "nonstandard stabilizers" and include compounds such as tall-oil based emulsions, bioenzymes, electrolytes, and pozzolans (*1*). Of the numerous conventional and nonstandard stabilizers available, most have shortcomings that limit their use to particular climate, soil, and traffic conditions.

In 1990, B.C. Chemicals Ltd. of Prince George, British Columbia, began producing tall-oil-based road-surface stabilizers. In a trial of one such product, a tall oil pitch (TOP) emulsion was applied to an unpaved 5.1-km section of the Willow Forest Service Road in the central interior of British Columbia in May 1992. The Forest Engineering Research Institute of Canada (FERIC) cooperated with the Prince George Forest District of the British Columbia Ministry of Forests (BCMOF) on a project to assess the emulsion's performance as a forest road-surface stabilizer.

The objectives of the project were to document site preparation and product application procedures and costs, and to evaluate the impact of the treatment on road-surface conditions and dustfall. A secondary objective was to formulate long-term cost projections for TOP emulsion treatment and selected alternatives.

Project and Site Descriptions

Tall oil, usually used as a raw material in the chemical industry (*2*), is manufactured from the soap skimmings generated by kraft pulping of resinous softwoods. If pulp mills cannot dispose of this material, the alternative is to burn it within the mill. Internal disposal is discouraged because it reduces pulping productivity and increases air pollution (H.S. Norman, General Manager, B.C. Chemicals Ltd., Prince George, British Columbia, personal communication, September 1992). B.C. Chemicals Ltd. has manufactured tall oil since 1975, and currently receives soap from five pulp mills in the central interior of British Columbia.

Because demand for the product is limited, B.C. Chemicals Ltd. explored alternative uses and markets for tall oil products, including road-surface stabilizers. In late 1990, B.C. Chemicals Ltd. began producing two chemically different compounds from tall oil; TOP and depitched tall oil. TOP is processed into an emulsion known as TOP emulsion. It is marketed for road-surface stabilization under the trade name Dustrol E. In 1993 some modifications were made to the original formulation of the TOP emulsion. This alternative product is marketed as a dust palliative under the trade name Dustrol EX.

TOP, dark brown in color, is soluble in organic solvents; it has excellent cementitious and waterproofing properties (*3*). TOP stabilizes an unpaved road surface by cementing aggregate particles together. This is in contrast to the extensively used calcium and magnesium chloride treatments, which are hygroscopic and stabilize a road surface by drawing moisture into the aggregate. The TOP emulsion used in the study contained a 25 percent concentration of pitch solids. Some preliminary applications were done in the region with concentrations of 40 and 50 percent, but thorough blending with the surfacing aggregate proved difficult to achieve.

There have been several trial applications of tall-oil-based emulsions in North America (*1*) using products from the southeastern United States. These are byproducts of the pine pulping process and are sometimes referred to as "pine tar derivatives" or "tree sap." Their chemical properties differ from B.C. Chemical Ltd.'s emulsions because a combination of both spruce and pine wood is used in the pulp manufacturing process in the Prince George area. Several cursory trials of tall-oil-based emulsions have occurred in the interior of British Columbia. In this study, FERIC monitored the performance of TOP emulsion (Dustrol E) only. A 5.1-km test section was established on a heavily traveled unpaved portion of the Willow Forest Service Road near Prince George. This route has a specified maximum gross vehicle weight rating of 82 t and is subject to off-highway log-hauling traffic. Road-maintenance costs are allocated proportionally to a group of industrial users (Willow-Ahbau Road Users Committee) based on the cumulative tonnage of products hauled over the road. Level grades predominate and the running surface averages 9 to 10 m wide.

Study Methods

Currently, no standard assessment procedure exists for evaluating road-surface stabilizers, although a variety of procedures has been proposed in the literature (*4,5*). After a review of various options, some procedures from UMA Engineering Ltd. were adapted for this project.

The application process was monitored to document techniques for mixing the emulsion with the aggregate, compaction procedures, emulsion transport and handling, and application costs. Visual assessments of road-surface conditions and dustfall were conducted at regular intervals from the time of application in May 1992 until the end of September 1992. Intermittent observations of road-surface conditions were made until May 1993. The 5.1-km treated section and an adjacent 5-km untreated section were monitored and compared.

One individual conducted all the assessments to ensure consistency in estimates of road-surface conditions and dustfall. Road-surface condition was evaluated for potholes, washboarding, surface ravelling, and ride quality for drivers. The frequency and severity of the surface condition indicators were ranked on a scale of 1 to 4, with preferred and severely deteriorated conditions represented by 1 and 4, respectively. Because conditions varied over the length of the test road, a weighted average was calculated for each observation date. The dust plumes created from passenger vehicles and log trucks were ranked on a scale of 0 to 5 against dust volume and driver visibility criteria. Dust-free and extreme dust conditions were represented by 0 and 5, respectively. Dust plumes generated by passing vehicles were photographed at two stations in the treated and control sections. Initially the assessments were conducted weekly, and then biweekly after the first 2 months of the trial.

Data about factors expected to influence product performance were collected. Grain size analyses were done on samples of the surfacing material. Local precipitation and temperature records were obtained from the Prince George Airport weather station, located 6 km from the study site. A traffic counter was installed for a 1-week period to provide estimates of traffic volume. Driving habits and speeds were noted during the regular site inspections. Records for the weight of logs and wood chips hauled by the user companies over the test section were also obtained. Finally road-maintenance procedures and costs for the treated and control sections were documented and described.

PROCEDURES FOR APPLYING TOP EMULSION

B.C. Chemicals Ltd. employed a contractor (Lobol Enterprises Ltd., Prince George, British Columbia) to apply the emulsion to the test road during two 12-hr periods on a weekend. A 25 percent water solution of TOP emulsion was specified for the trial. Two tanker trucks with capacities of 16 000 L and 35 000 L and one quad-axle tanker trailer carrying 19 000 L were used to transport a 50 percent solution to the work site from the Prince George plant. Water was added at the site to dilute the emulsion to the required concentration. The tankers, equipped with spray bars (Figure 1), applied the emulsion to the road surface and refilled from the trailer before returning to the plant 23 km away to reload.

Two motor-graders prepared the road surface and blended the emulsion into the aggregate. One grader, equipped with carbide-tipped picks on the cutting edges, worked ahead of the application equipment, recovering material from the road shoulders and blading the surfacing aggregate into two windrows, one on each side of the road. The second grader, equipped with straight cutting edges, retrieved a portion of material from the windrow and spread a thin layer of aggregate in preparation for each spray coat of emulsion (Figure 2). To minimize disruption of traffic, the application was completed on successive subsections approximately 1 to 1.3 km long.

FIGURE 1 Emulsion application.

FIGURE 2 Motor grader spreads aggregate before each spray coat.

Several coats of emulsion were required to reach the targeted surface layer thickness of 5 to 7 cm. To bond the surface layer with the underlying base course, the first coat of emulsion, or primer coat, was sprayed onto the exposed base course at a rate of 1 L/m^2. Production coats were applied at 2 L/m^2 to successive 2-cm layers of surfacing aggregate until the desired surface layer thickness was reached. An average accumulation of 8 L/m^2 was planned; however, the actual rate varied from 8.5 to 12.4 L/m^2 depending on subsection requirements. Some subsections appeared to contain a larger proportion of fines in the aggregate and thus required more emulsion to achieve complete aggregate coating. In total, 453 000 L of emulsion was applied to the 5.1-km test road, resulting in an average application rate of 9.4 L/m^2.

A multiwheel pneumatic-tire roller completed the application process by kneading and compacting the emulsion-coated aggregate. The kneading action produced by this type of compactor helps create a more stable, less permeable surface layer. As with other phases in the process, procedures for compaction were developed empirically and no density criteria were specified. The surface was rolled repeatedly until firm and smooth. Initially some of the emulsion-coated aggregate stuck to the compactor's tires, but this was alleviated by adding 1000 kg of water ballast to the machine and by reducing its tire pressure. Vibratory steel drum rollers are unsuitable for this application because pitch-coated aggregate sticks to the drum.

The manufacturer established a unit cost of C$0.10/L (mid-1992) for the TOP emulsion product, including application, for roads within British Columbia's central interior region. If the target application rate of 8 L/m^2 can be sustained, the cost to stabilize the surface on a 9- to 10-m-wide forest road is approximately $7,000/km for the complete process.

Results and Discussion

Monitoring Performance of TOP Emulsion

The results of performance monitoring are summarized in Figures 3 and 4. Inferences about TOP emulsion performance can be made from trends shown in the charts. When comparing observations of the treated and untreated road sections, note that the untreated section was graded regularly and received lighter traffic volumes. No grading occurred on the treated section during the study period.

Four indicators of road-surface condition are presented in Figure 3. Light potholing was observed over some subsections of the treated road. Potholes formed in groups, oriented longitudinally, and were attributed to road surface geometry, drainage, and how well the emulsion blended with the aggregate. Early in the trial, potholes were patched by hand with an aggregate-emulsion mixture. Potholes also developed on the untreated section during the summer and were repaired by regular blading. In late summer and fall, and coinciding with higher rainfall levels, the incidence and severity of potholing were higher on the untreated road although this section was subject to regular maintenance.

Washboarding of the road surface before treatment was a recurrent problem even on level grades and tangent road sections. Performance monitoring showed that washboarding was eradicated with the application of TOP emulsion. Washboarding continued to be a problem on the untreated section, although some observation dates coincided with blading maintenance and resulted in a low rating.

Surface ravelling, and the resulting accumulations of float gravel on roadsides and between wheel tracks, is a condition that was effectively controlled on the treated section. The incidence of surface ravelling on the

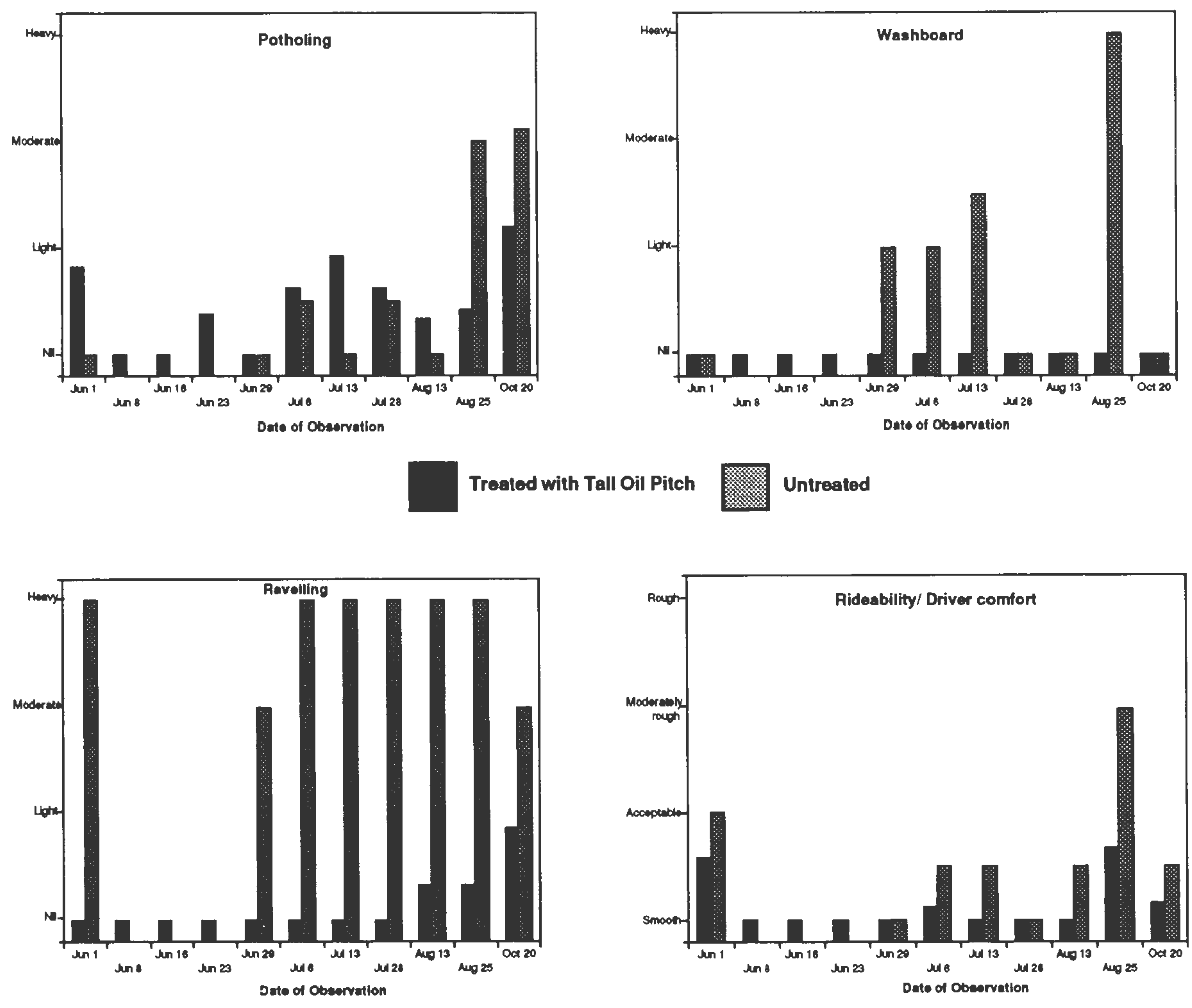


FIGURE 3 Results of performance monitoring.

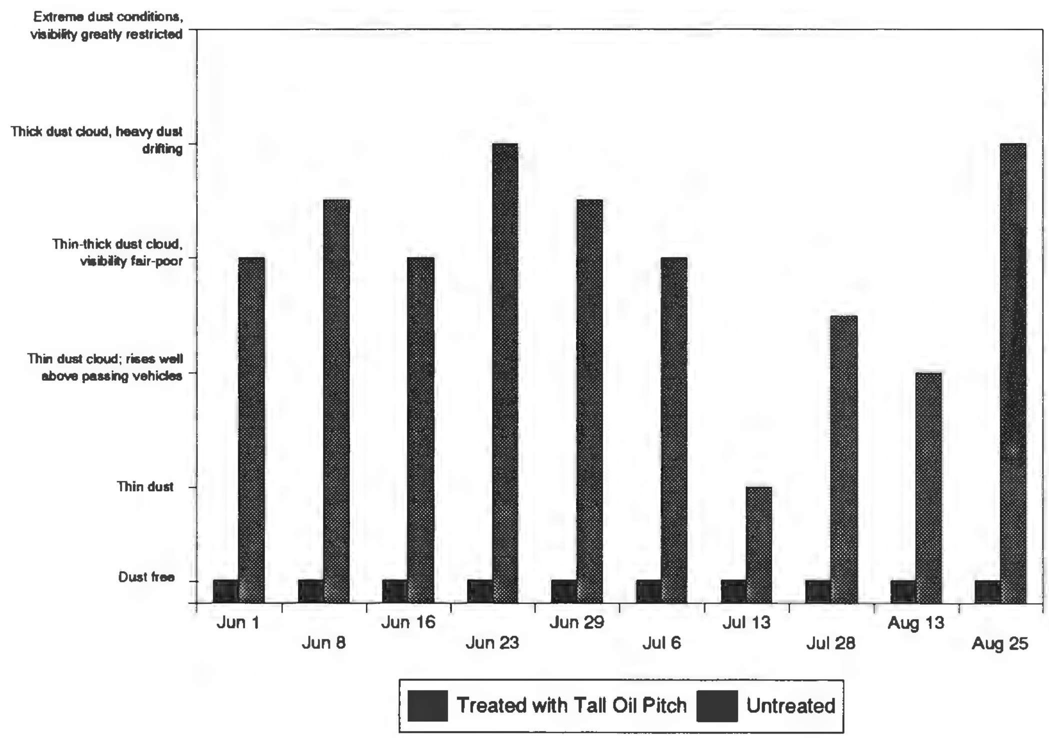


FIGURE 4 Dustfall observations.

untreated road remained high and was especially acute on curves.

The fourth indicator of surface condition, rideability or the level of driver comfort experienced while traveling on the road, is a subjective evaluation often used as an indicator of surface quality. The quality of ride for the treated section was usually smooth and superior to that for the untreated road. This opinion was corroborated by FERIC in interviews with many regular users of the road.

The study showed that dustfall was consistently and effectively controlled on the TOP-treated road (Figure 4). Maintenance managers found that industrial and public users appreciated the dust-free driving conditions. Figures 5 and 6 illustrate the differences in dustfall generated by vehicles passing over the treated and untreated sections on the same day.

In June 1993, after more than 1 year of service, strips of potholes had formed over much of the road surface of the treated section. Rather than attempting repairs, much of the treated section was ripped up and reshaped with a motor grader. However, the surface of one 500-m subsection remained remarkably smooth and well stabilized and was left intact (Figure 7).

FIGURE 5 Treated road surface.

FIGURE 6 Adjacent untreated road section.

FIGURE 7 Surface performing well after 14 months of treatment.

Grain Size Analysis

A key factor influencing road-surface stability is surfacing aggregate gradation. Aggregate with an insufficient fines fraction (i.e., material passing a 0.074-mm sieve) will not have the necessary binding properties; conversely, a large proportion can retain too much moisture, thus leaving the surface susceptible to frost damage. Acceptable results for any treatment should not be expected unless gradation has been analyzed and adjusted to meet specifications.

The grain size chart in Figure 8 shows the grain size distributions for two samples taken from material windrowed during the application process. The samples fall within the British Columbia Ministry of Transportation and Highways' acceptable range for surfacing aggregate (represented by the shaded area). The proportion, by weight, of fines in the two aggregate samples was 5.0 and 5.7 percent. Nine months before application, six samples were retrieved from ravelled material accumulated on roadway shoulders. Fines content for these samples ranged from 9.2 to 18.4 percent. Although not representative of the surface layer subsequently formed and treated with TOP, the results suggest that variations in gradation existed over the road. This finding concurs with the contractor's experience during application: the aggregate on some subsections had a high proportion of fines and was difficult to coat totally with emulsion.

Climatic Conditions

Climatic conditions were extraordinarily challenging to the performance and lifespan of the treatment. Figures 9 and 10 summarize precipitation and temperatures during the study period. Total precipitation, including

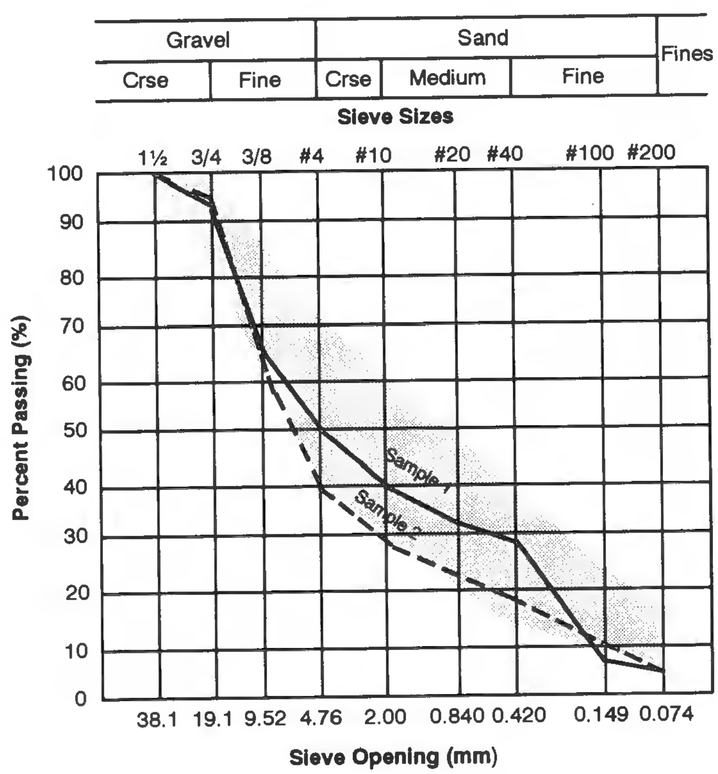


FIGURE 8 Gradation of surfacing aggregate.

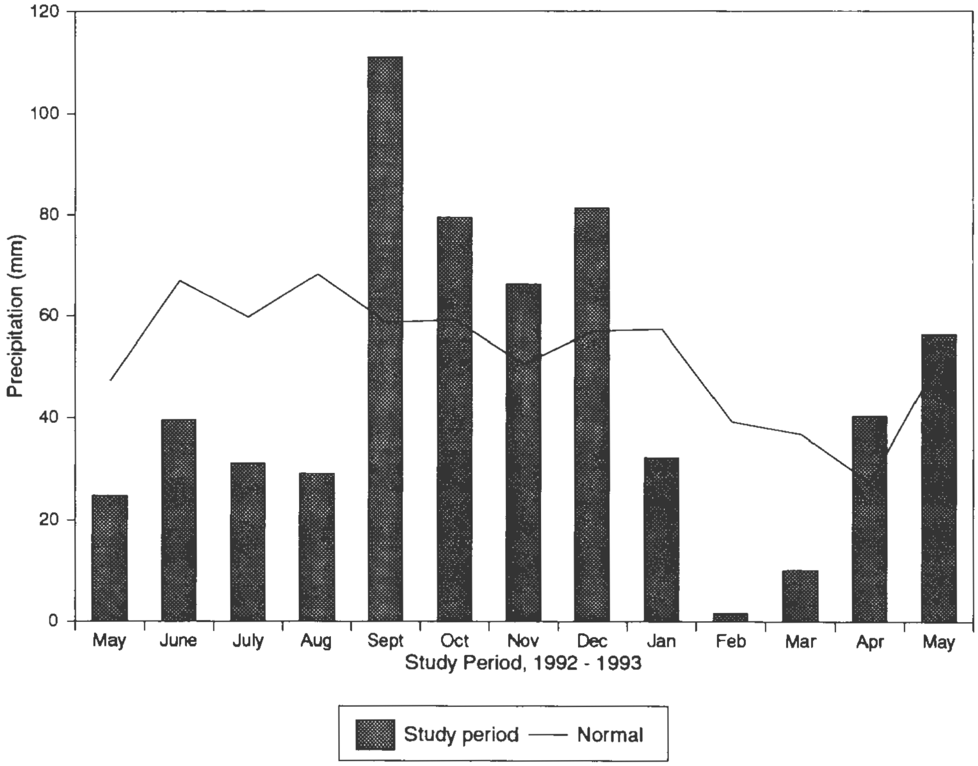


FIGURE 9 Precipitation record.

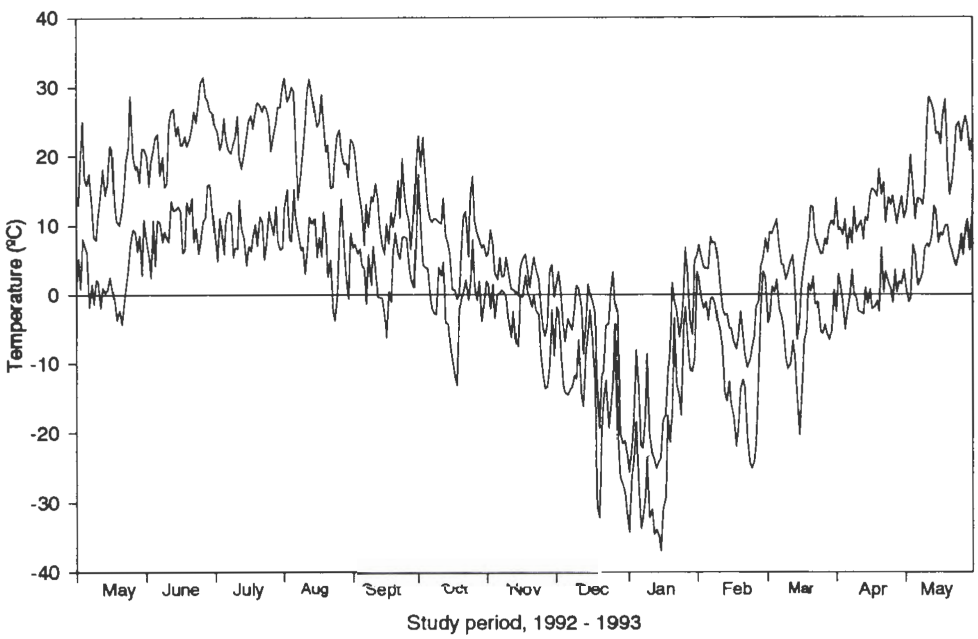


FIGURE 10 Daily maximum and minimum temperatures.

rainfall and snowfall, for the 12-month period of May 1, 1992, to April 30, 1993, was 548 mm. The pattern of precipitation varied considerably from the pattern normally experienced in the area. The summer was unusually dry and preceded a wetter-than-normal fall. The unusually wet fall period strongly tested stabilizer performance because the propensity for potholing and product leaching increased in these weather conditions.

Snowplowing usually leaves a compact snow surface, which is sanded to provide traction for vehicles. However, snowfall accumulations in January, February, and March were below normal. As a result, during the winter the treated road was usually exposed, leaving it subject to precipitation, temperature fluctuations, traffic, ice blading, and sanding. In the absence of a protective snow layer, it is probable that the quality of treated surface continued to deteriorate during the winter, thus shortening treatment life.

Temperatures ranged from +32 to −37°C during the study period (Figure 10), and numerous fluctuations above and below freezing occurred during the transition periods (spring break-up and fall freeze-up). In British Columbia's central interior there are usually summer and winter log-hauling seasons separated by 2- to 4-week intervals of freeze-up and break-up in the spring and fall. During these transition periods the road surface is more susceptible to damage from heavy traffic because the strength of the underlying base course and subgrade is reduced.

Traffic Conditions

Three sawmills, one wood remanufacturing plant, and two aggregate and asphalt production facilities contribute to the heavy, multiaxle vehicle traffic on the treated and untreated road sections. These vehicles include on- and off-highway log trucks, B- and super B-train chip vans, lumber carriers, gravel trucks, and many other types of heavy industrial carriers. Plant workers and local residents contribute to light passenger vehicle traffic. Undoubtedly the control of dust during dry periods contributed to the excessive driving speeds observed on the treated road.

Table 1 summarizes traffic data from an automatic hose count system for the period of August 28 to September 3, 1992, inclusive. For an unpaved forest road, a very high volume of heavy industrial and light passenger vehicle traffic was experienced during weekdays. Log-hauling traffic fluctuated depending on the season and on the location of the mill's log sources. During spring break-up and fall freeze-up, log hauling was suspended until road conditions stabilized. However, the treated road sustained continued use by chip vans, lumber carriers, and other heavy vehicles throughout these periods when the surface was prone to damage.

Records from Willow-Ahbau Road Users Committee indicate that approximately 650 000 t of logs, wood chips, and gravel were hauled over the test road section from May 1, 1992, to April 30, 1993. Weights of other products, including lumber, were not available. FERIC estimates that the tonnage for recorded materials alone translates into more than 15,000 one-way trips. These figures concur with FERIC's observations that the road sustains very high volumes of heavy industrial and light passenger traffic.

Maintenance Procedures

After the TOP emulsion was applied and follow-up patching completed, no blading maintenance with motor graders was done on the treated section during the 1-year evaluation period. Although it becomes increas-

TABLE 1 Traffic Volume: August 28 to September 3, 1992, Inclusive

Type of vehicle	Average daily traffic[a] Weekdays (no. vehicles)	Weekends (no. vehicles)
Light passenger vehicles	310	189
Heavy industrial multi-axle vehicles	402	35
Total	712	224

NOTE: Summarized from Carol L. Smith, P.Eng., Transtech Data Service (1992), Victoria, British Columbia; unpublished report, September 1992.

[a]Total traffic volume obtained from hose count installation. The breakdown between light passenger and heavy industrial traffic was estimated from industrial user records.

ingly malleable in hot weather, the emulsion-coated aggregate forms a consolidated layer that cannot be rebladed. Before being treated, the road often required twice-weekly grading. One occasion warranted full grading with four to six passes to complete a road section, and the other occasion usually required light patch-grading. The lower traffic volume on the untreated section resulted in a grading frequency of approximately once per week.

Supervisory personnel noted that without the TOP stabilization treatment, the dryer-than-normal summer conditions would have necessitated daily watering of the road surface to control dust, thus adding significantly to road-maintenance costs (C. Andreschefski, Area Supervisor, Canadian Forest Products Ltd., Netherlands Division, Prince George, British Columbia, personal communication, September 1993).

Other Observations

Identification of the physical and chemical processes that define the soil-emulsion interaction is a necessary first step in the development of superior emulsion formulations and application procedures for road-surface treatment. B.C. Chemicals Ltd., in collaboration with Arbokem Inc. of Vancouver, British Columbia, is conducting research to determine how tall-oil-based emulsions interact with the surfacing aggregate and how to accelerate the curing process.

TOP emulsions require 1 to 3 days to cure, depending on weather conditions. A faster curing rate is needed to allow normal traffic flow to resume promptly. Immediately following application, vehicle tires will often pick up some of the emulsion-coated aggregate and deposit it on vehicle surfaces. Some drivers become annoyed if they are not forewarned about these conditions because emulsion-coated aggregate is difficult to clean from vehicle surfaces. Once the precise curing mechanism is identified, emulsion additives may be proposed that can speed up the process.

Tests have shown that the leachate of road materials treated with TOP emulsion is nontoxic (H.S. Norman, General Manager, B.C. Chemicals Ltd., Prince George, British Columbia, personal communication, September 1992).

Thorough blending of the emulsion with the aggregate is the most challenging aspect of product application and requires repeated passes with a motor grader and spray truck. One possible explanation for the difficulty of mixing is that the pitch solids in the emulsion may be intercepted by the upper surface of the aggregate layer, where they accumulate. Initially each spray coat appears to saturate the aggregate. However, if the pitch component of the emulsion is filtered out at the surface, only dark-colored water penetrates through to the lower portion of the aggregate layer. Thus the product must be applied over several thin lifts of aggregate.

The hot, dry weather during the application period caused the water in the aggregate to evaporate quickly. Sometimes light- colored streaks in the aggregate, indicating low pitch content, were revealed after the surface layer dried. This puzzling phenomenon was perhaps due to an uneven distribution of TOP within the emulsion after water was added on site. Also, high interfacial tensions between the emulsion and soil particles may be a contributing factor (H.S. Norman, General Manager, B.C. Chemicals Ltd., Prince George, British Columbia, personal communication, May 1994). When the streaks appeared, the surface was reworked with the motor grader and additional emulsion was applied.

TOP emulsion treatment is not intended to create a permanent road surface. As expected, some road sections gradually deteriorated during the year-long study. Three processes of the surface failure were observed. The predominant type of failure was potholing, and predictably it was observed most often on sections with negligible crowning and poor drainage. Another failure process was spalling of the fine emulsion matrix at the surface. This was not a serious concern and occurred only intermittently. Interestingly, potholing or spalling failures were not associated with vehicle wheel tracks. The third failure process was deformation of the surface in the wheel tracks. It occurred sporadically and was observed on the road sections that remained saturated for a lengthy period following application. TOP emulsion had been applied to these sections late in the day, under shaded conditions, when the rate of evaporation had slowed. The surface and base course likely did not develop sufficient strength before regular traffic resumed.

Several sections of the test road performed extraordinarily well during the trial. In particular, one 500-m section, including a curve, remained in excellent condition beyond the year-long study period and was used through a second summer hauling season without any maintenance. Interestingly, this road section had received the lowest application rate of emulsion, at 8.5 L/m^2. However, this 500-m section was also the subject of a cursory TOP trial one year prior to this study. A 40 percent concentration of TOP was lightly applied at that time.

The superior performance of this 500-m section suggests that residual amounts of pitch persisted from the cursory trial and that the effects of tall oil-based emulsion applications may be cumulative. It is likely that another contributing factor was the well-drained road base on this particular section.

Alternative Treatments

Expenditure profiles over a common 5-year planning period were prepared for three alternatives: Dustrol E, Dustrol EX, and no treatment. Costs are estimated for the 5-km-long, 9-m-wide test road and are thus site specific. The first year of profiles is shown in Table 2. A summary of the annual costs and the expenditures by work phase is presented in constant-worth dollars in Tables 3 and 4, respectively. The present worth of each expenditure profile was calculated. Information about the Dustrol EX alternative was collected through discussions with users and visits to several roads where the product was being tested in north central British Columbia. The maintenance regime for the no-treatment option was derived from the road-maintenance records of the Willow-Ahbau Road Users Committee.

Dustrol E is marketed as a road-surface stabilizer, and Dustrol EX as a dust control agent. Perhaps emphasis on the latter distinction is unwarranted because reduced dust emissions are the result of a well-stabilized road surface. The two products provide different degrees of surface stabilization. The depth of stabilization within the surface, the chemical properties and process of interaction with the aggregate, and the application techniques vary between the two products.

At present, a more rigorous procedure, and therefore a more expensive one, is being used to apply the Dustrol E product than the Dustrol EX product. The initial cost of Dustrol EX treatment is considerably less because it is usually applied to a road surface with little preparation in only one or two spray coats. Dustrol EX is applied at a rate of 3 L/m^2 and costs $2,800/km for a 9-m-wide road. Observations indicate that only a thin upper layer of the road is treated and that surface deterioration occurs more rapidly than the road treated with Dustrol E. It is likely that a mid- to late-summer reapplication of Dustrol EX would be necessary on the heavily used industrial road chosen for this study. Additional costs for surface patching and grading can be anticipated for the Dustrol EX option and are included in the cost profiles. The no-treatment alternative includes the cost of watering to reduce dust during the summer months and the cost of resurfacing to replace aggregate lost after five years of heavy traffic and continual blading activity.

TABLE 2 Estimated Expenditure Profiles for Alternative Treatments (Year 1 of 5-Year Planning Period)

		Dustrol E Emulsion[a]		Dustrol EX emulsion		No treatment	
End of year	End of month	Work performed	Constant worth ($/km)	Work performed	Constant worth ($/km)	Work performed	Constant worth ($/km)
0	May	Application	7000	Application	2800	Grading	500
	June	Patching	80	Patching & grading	240	Grading	500
	July			Grading	160	Watering & grading	1000
	August	Patching & grading	160	Re-application	2000	Watering & grading	1000
	September			Patching & grading	200	Grading	500
	October					Grading	500
	November	Grading	160	Grading	160	Grading	500
	December						
	January						
	February						
	March	Grading	160	Grading	500	Grading	500
	April	Grading	160	Grading	500	Grading	500
1	May	Application	7000	Application	2800	Grading	500

NOTE: Road maintenace costs in December, January, and February are omitted because they are common to all the alternatives. Estimates apply to the 9-m wide test road section monitored in the study.

[a]Assume that residual TOP will accumulate in the surface layer, thus reducing application costs to $4000/km in Years 2 to 5.

TABLE 3 Estimated Annual Road-Surfacing and Maintenance Expenditures for Alternative Treatments

End of year	Dustrol E emulsion ($/km)	Dustrol EX emulsion ($/km)	No treatment ($/km)
0	7 000	2 800	500
1	7 720	6 560	5 500
2	4 720	6 560	5 500
3	4 720	6 560	5 500
4	4 720	6 560	5 500
5	4 720	6 560	25 000
Present worth	28 569	29 499	36 365

NOTE: Road maintenance costs common to all the alternatives are omitted. The assumptions for the analysis are: 5-year planning period, 8% discount rate, and monthly compounding. The measure of effectiveness is the present worth method applied to constant worth, before-tax cash flows. Estimates apply to the 9-m wide test road section monitored in the study.

TABLE 4 Estimated Total Expenditures over 5-Year Planning Period by Maintenance Activity

Work performed	Dustrol E emulsion ($/km)	Dustrol EX emulsion ($/km)	No treatment ($/km)
Annual product applications	30 000	16 800	-
Re-applications within each year	-	10 000	-
Regular grading	2 800	7 100	22 500
Surface patching	800	1 700	-
Watering for dust control	-	-	5 000
Resurface with new aggregate	-	-	20 000
Present worth	28 569	29 499	36 365

See footnote for Table 3.

Analysis shows that the treatment alternative with the largest initial application costs may prove to be the most cost-effective over an extended planning period when ancillary costs and the time value of money are considered. When assessments of nonmonetary factors (such as the quality of the surface created and the improvement to road safety) are made, the Dustrol E alternative compares favorably with other options.

Conclusions

The test road treated with TOP emulsion performed well during the year-long study, and local road users were pleased with the results. Climatic conditions and traffic patterns were very challenging for product performance, yet a maintenance-free surface was retained during the year-long study. One section in particular performed extraordinarily well, pointing to the good potential of TOP emulsion for road-surface stabilization. An advantage of TOP emulsion over some alternative nonpermanent treatments is its ability to act by cementing aggregate particles together, thus forming a water-resistant surface. Because it is not a hygroscopic agent, performance can be maintained over extended dry periods. TOP emulsion potential would be enhanced if practical repair procedures could be developed to enable patching of isolated sections of potholing while leaving the acceptable road surfaces intact.

The relatively high initial costs and difficulties in application demand that potential users conduct long-term economic analyses to determine the benefits of using TOP emulsion in a road-surface stabilization program. When analyzing alternatives with more attractive start-up costs, users are reminded that overall maintenance costs are likely to decrease and the quality of the surface produced is likely to improve as the level of initial investment is raised.

Note that two chemically different tall-oil-based emulsions are produced in British Columbia's central interior and marketed as road-surface treatments: Dustrol E and Dustrol EX. Application procedures for the E and EX emulsions have developed through a process of trial and error. Detailed scientific study is required to refine formulations and application methods and to produce more consistent results. B.C. Chemicals Ltd. has undertaken important research to better understand how the locally produced tall-oil-based emulsions stabilize unpaved road surfaces. Without information about the product's interaction with different soil types and the process of curing the product when it is applied to the road surface, it will be difficult to improve product performance and expand on the potential for commercialization.

Preparation of the road surface is perhaps the key phase of a stabilization program. Correctly graded aggregate must be in place and properly shaped to form a well-crowned road surface. Positive drainage over an entire road surface is critical to maintaining it. Blading practices affect surface quality, and should be monitored to ensure that the geometry of the road surface is preserved. The benefits of applying a chemical stabilizer will not be fully realized unless resources are invested in preparation.

Forest-road-maintenance managers will sometimes be skeptical of a nonstandard stabilizer until it can demonstrate clear benefits and consistent performance. Although some product formulations may be proprietary, it is incumbent upon producers and marketers to describe the nature of their product and explain how it will work to stabilize a road surface. With the proliferation of surface treatment compounds, road-maintenance managers need concise and complete information about a product to decide if an investment is warranted.

It is often difficult for road-maintenance personnel to choose the most cost-effective product or decide whether trial of an alternative is warranted. The decision remains a site-specific one. A product must be matched to local climate conditions, type of traffic, and the type and gradation of surfacing material. The best decision will result from a comparison of alternatives over a planting period of several years, coupled with the sound judgment of experienced managers concerned with both the quantitative and nonquantitative aspects of investment alternatives (6).

ACKNOWLEDGMENTS

The authors would like to thank Jeff Kotowsky of the British Columbia Ministry of Forests for his assistance in collecting data during the project.

This paper was originally published as Technical Note TN-220 by the Forest Engineering Research Institute of Canada (FERIC), Vancouver, British Columbia.

REFERENCES

1. Scholen, D. E. *Non-Standard Stabilizers.* FHWA FLP-92-011, Final Report. FHWA U.S. Department of Transportation, 1992, 113 pp.
2. McSweeney, E. E., H. G. Arlt, Jr., and J. Russell. *Tall Oil and Its Uses-II.* Pulp Chemicals Association, Inc., New York, 1987, 132 pp.
3. Mazuch, L., J. Sam, and L. Maco. *Laboratory and Field Evaluation of Tall Oil Pitch Modified Asphalt Cement.* British Columbia Ministry of Transportation and Highways, Geotechnical and Materials Engineering, Victoria, British Columbia, Canada, 1990, 27 pp.
4. Aquin, R. C., et al. *Single Surface Treatment or Dust Control on Gravel Roads?* Report No. MI-148. Ontario Ministry of Transportation, Engineering Materials Office, Bituminous Section, Toronto, Ontario, Canada, 1991, 24 pp.
5. UMA Engineering Ltd. *Guidelines for Cost Effective Use and Application of Dust Palliatives.* Roads and Transportation Association of Canada, Ottawa, Ontario, Canada, 1987, 87 pp.
6. Szonyi, A. J., et al. Chapter 4: Comparison of Alternatives. In *Principles of Engineering Economic Analysis, Canadian Edition,* John Wiley & Sons, Toronto, Ontario, Canada, 1982, pp. 119–187.

Calcium Chloride in Road Construction

Pirjo Kuula-Väisänen, Hanna-Liisa Järvinen, and Pertti Nieminen,
Tampere University of Technology, Finland

The effects of calcium chloride ($CaCl_2$) on the properties of Finnish road construction materials used in the base course and subbase were examined. The properties studied in laboratory conditions were, first, the basic properties of grain size, specific surface area, water adsorption, and zeta-potential; and second, the compactability, frost heave and migration of these materials. Studies were also performed on full-depth reclamation with $CaCl_2$ used in the United States and Canada. The literature review yielded a positive indication for the use of $CaCl_2$ in road construction. The materials studied were crushed aggregates and glacial tills. The test results indicate that $CaCl_2$ enriches in finer parts of materials, binds the finer particles together, and increases their water adsorption. Changes in the basic properties affect compactability and frost heave. The compaction needed to achieve a maximum dry unit weight decreases with $CaCl_2$ use, and the material stays wet for long periods. When $CaCl_2$ was used as an additive, the compactability of materials at below-zero temperatures was clearly better and the dry unit weight was nearly the same as at room temperature. Frost heaves decreased remarkably, even by 80 percent in frost-susceptible materials. The migration studies showed that the stability of $CaCl_2$ is poor when it is in contact with percolating water. Despite the positive technical facts, the environmental effects should be considered carefully. The environmental effects were not studied, but some estimations were made, mainly based on a literature review.

The Finnish National Road Administration (FinnRa) and Finnish cities have used flake calcium chloride ($CaCl_2$) for years to aid in dust control. The use of calcium chloride in maintenance is based on its beneficial chemical properties. Calcium chloride is hygroscopic and absorbs moisture from the air. Because of its deliquescence, $CaCl_2$ can dissolve in the presence of moisture to form a liquid solution. The vapor pressure of this solution is significantly lower than that of pure water, and surface tension is higher. The properties of $CaCl_2$ make it evaporate from solution much more slowly than from pure water. Dissolution of $CaCl_2$ in water is an exothermic reaction.

Finland has many low-volume roads, with several problems associated with their maintenance and construction. For example, frost heaves in springtime cause restrictions for traffic and may prevent driving for weeks. The lack of sufficient materials also has led to a need for new materials and methods for base course construction. The chemical nature of $CaCl_2$ could make it a beneficial additive to increase the stability of various existing construction materials. Partly due to these problems and lack of knowledge about $CaCl_2$, FinnRA and Kemira Chemicals, Ltd. initiated a project with a literature review and laboratory research on the use of $CaCl_2$ in road construction. The study was carried out by the Institute of Engineering Geology at Tampere University of Technology.

The literature review included articles and research reports published in the United States and Canada.

Three main applications were mentioned for $CaCl_2$ in road construction: compaction at above and below freezing temperatures, protection from frost heaving, and full-depth reclamation (FDR). All of these could have significance in Finland. The laboratory research to discover how $CaCl_2$ affects the basic and technical properties of Finnish road construction materials used in base course and subbase was based on the facts found in the literature.

Literature Review

Below-freezing temperatures cause problems in obtaining specific densities because of the formation of ice within the pore spaces. Haas et al. (*1*) found that calcium chloride can significantly improve the compactability of frozen soil. The test was performed at −7°C and the silty sand was treated with 2 and 3 percent $CaCl_2$. The effect of $CaCl_2$ on compactability was also tested at room temperature. The dry density of soil increased only slightly when compacted with 1 percent of $CaCl_2$.

Slesser (*2*) and Slate (*3*) investigated the effect of $CaCl_2$ on the formation of ice lenses in subgrade soils. Both studies indicated that a few percent of $CaCl_2$ can reduce frost heave noticeably. According to Slesser, 2 to 3 percent of $CaCl_2$ can eliminate frost heave from coarser material. Fine-textured material would need 5 to 7 percent of $CaCl_2$. Slate suggested adding 2 percent $CaCl_2$ to silt, 1 percent to clay, and 0.5 percent to coarser material.

Reckard (*4*) examined the effect of $CaCl_2$ on frost heave and residual heave. In his experiments a schist material represented common soil types often used locally in road construction. The test results indicated that even small amounts of $CaCl_2$ may greatly decrease frost heave as well as residual heave. The results suggest that the freezing point was depressed by $CaCl_2$ and material was less frozen with higher salt content. In addition, the results indicated that there might be another mechanism influencing reduction in frost heave. In samples containing more salt, there was a greater delay between the time that freezing temperatures were initiated and the time that heaving began. Reckard also stated that the long-term heave rate (weeks or months) is still unknown.

FDR with $CaCl_2$ has been applied for low-volume secondary roads requiring rehabilitation in the United States and Canada (*5*). In this process the existing bituminous surfaces are pulverized and blended with a predetermined amount of granular base material. Liquid $CaCl_2$ is added twice: after pulverization and after the material is repulverized, graded, and rolled. The new road surface is allowed to cure for several weeks. The use of this reclamation technology with $CaCl_2$ seems to be based on the good results in field performance rather than on research results.

Slate (*3*) also investigated the migration of $CaCl_2$ under field and laboratory conditions. The research results indicated that lateral migration was slower than vertical migration. $CaCl_2$ should not be applied more deeply than necessary because the contact with groundwater will accelerate its migration. According to Slesser's field and laboratory studies (*2*), sodium chloride (NaCl) was more susceptible to lateral surface washing than $CaCl_2$. A bituminous pavement decreased migration of both salts in fine-grained soils. The pavement prevented the leaching of $CaCl_2$ by percolating water as well as surface-washing of NaCl. A high groundwater table accelerated the lateral movement of both salts. In sandy clay calcium cations are more permanent than sodium cations, and the two cations are more permanent than chloride anion.

It is not known to what extent $CaCl_2$ leaches through the embankment over time, nor how long the lowering effect on frost heave will last. Reckard (*6*) estimated the migration of $CaCl_2$ by applying half-life calculations and assuming that salt dissipates exponentially. Because of difficulties in determining background levels and variations in the test road conditions, the calculated data are inconclusive. However, the field data indicated that very little, if any, salt remains in road embankments several years after application.

Shepard et al. (*5,7*) investigated residual chloride levels in the test road section that was rehabilitated by FDR with $CaCl_2$. The test results indicate that in FDR, $CaCl_2$ is retained effectively in the road structure during the first 10 months. The residual chloride results revealed that after the second and third year, $CaCl_2$ had disappeared.

Calcium is essential for plant growth, but high concentrations of calcium will cause excessive salinity and may be specifically toxic. Calcium is added to the soil for pH control and to replace excessive sodium (*8*). Calcium diminishes plants' ability to absorb sodium, which prevents the negative effects of sodium. The studies indicate that chloride has an especially toxic effect on plants (*9*). There is currently no definite information on the effects of $CaCl_2$ on plants. Many other factors (*9*), such as changes in weather and plant diseases, can cause the same kinds of damage as salts.

Materials and Methods

Six different types of materials were chosen for the laboratory tests. Three were crushed aggregates that were originally rock, gravel, and glacial till (Figure 1). These kinds of aggregates are usually used in road construc-

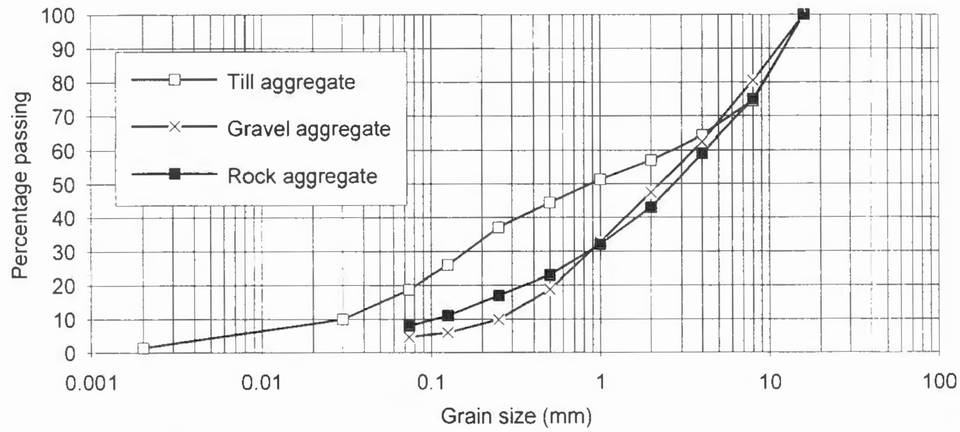


FIGURE 1 **Grain size distributions of crushed aggregates.**

tion in Finland. According to the Unified Soil Classification System (USCS), the gravel and rock aggregate are well-graded sands (SW) and the till aggregate is silty sand (SM). The mineralogical compositions of the samples were typical to the Finnish aggregates. Samples contained varied amounts of quartz, plagioclase, and feldspar and small amounts of biotite. The rest of the materials were glacial tills chosen for their different frost heave properties (Figure 2). According to the grain size distributions, tills could be classified as SM according to the USCS, although this kind of classification method does not describe the whole nature of glacial tills. Till Number 1 was a highly frost-susceptible, weathered till from northern Finland that contained iron. The two other tills were typical glacial tills with different fines contents (Number 2 had about 30 percent and Number 3, about 10 percent). The mineral composition of these two was typical to Finnish tills, that is, mainly quartz, plagioclase, and feldspar. The bituminous pavement used in FDR was obtained from a highway and crushed with a laboratory crusher. The amount of bituminous pavement in FDR samples was 25 percent by weight of the aggregate. The $CaCl_2$ solution was formed from flakes (77 percent). The solution used in the tests was 32 percent, and the amount used was 1 percent by weight of soil calculated as 77 percent $CaCl_2$.

The chloride content of the samples was measured with a chloride-selective electrode from a dispersion containing 70 g of aggregate and 150 mL of distilled water. When the chloride content of finer fractions was measured, the amount of sample was naturally smaller and the amount of water was decreased accordingly. The $CaCl_2$ content of the materials was measured before and after the tests.

In order to find out the effects of $CaCl_2$ on basic properties of materials, four variables were chosen: grain size, specific surface area, water adsorption index, and zeta potential. Grain size distribution was measured by sieving, and the grain size of fine fractions ($d < 0.074$

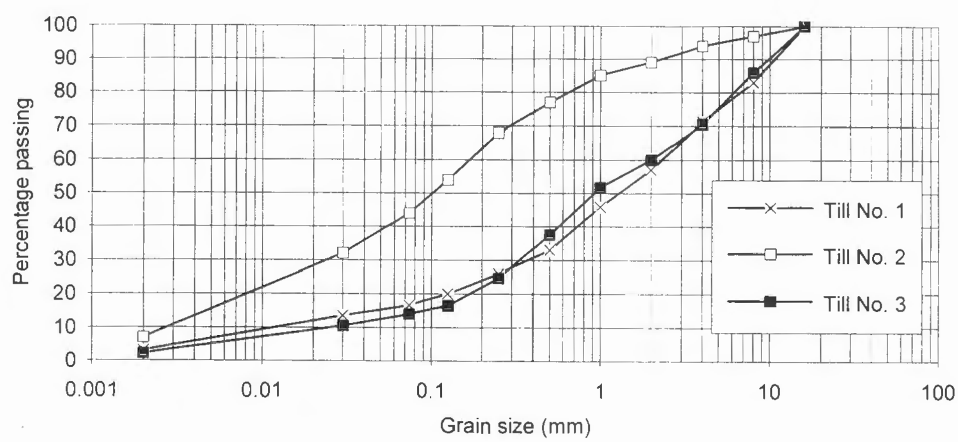


FIGURE 2 **Grain size distributions of glacial tills.**

mm) was determined with a laser diffraction analyzer. Specific surface area is the cumulative particle area per weight or volume unit (m^2/g or m^2/cm^3). Grain size distribution, grain shape, and porosity of grains affect the value of a specific surface area. The specific surface area values have proven important, for example, in evaluating the weathering sensitivity of aggregates (*10*). For normal aggregates the values are between 0.5 and 5 m^2/g, although the value varies from case to case. Specific surface areas of fine fractions were measured with the nitrogen adsorption method with Micromeritics Flowsorb. Water adsorption indices were determined using a method developed in the Institute of Engineering Geology at Tampere University of Technology (*11*). With this method the amount of water adsorbed on an oven-dry sample during a week in a desiccator in 100 percent relative humidity is determined. The water adsorption index is the amount of water adsorbed expressed by percentage of the dry weight of the sample. Normally the water adsorption values are between 0.5 and 2.0 percent. Zeta potential is an experimental value of the charge of a particle in a liquid. The value is expressed in millivolts and the value describes, among other things, the flocculation strength of particles in dispersion. The normal zeta-potential values for aggregates in distilled water are −20 . . . −30 mV. Zeta potentials were measured in distilled water with Zeta Meter System 3.0.

Compaction studies were performed with a gyratory compactor developed in Finland. The compaction effort used can be observed during measurement. The diameter of the compacted specimen is 10 cm and the height of the specimen is 10 to 13 cm. Before the effect of $CaCl_2$ was studied, the optimum water content and the maximum dry unit weight of the aggregates were determined. When $CaCl_2$ was used, 1 percent liquid $CaCl_2$ and an optimum amount of water was mixed with the material, and the compaction was started immediately. When the compactability of frozen materials was studied, materials were mixed with an optimum amount of water and frozen overnight at −10°C. Evaporation of water was prevented with a plastic bag. One percent of liquid calcium chloride was added, the sample was mixed, and the compaction was started immediately. The compression strength of specimens compacted with the gyratory compactor was also measured.

The frost-heave tests were performed with equipment developed at the Institute of Engineering Geology at Tampere University of Technology (*12*). The set-up for tests was the so-called "open system" in which water flows through the unfrozen portion of the sample into the freezing front, and the sample is frozen from the bottom. Materials were mixed with water containing liquid $CaCl_2$ one night before the test was begun. Samples were compressed under a strength of 38.5 MPa overnight, and plain water or water with $CaCl_2$ was conducted into the sample during compression. The test contained two freezing steps: the temperature of the freezing liquid was initially −10°C, decreased to 20°C after a day. The frost heave and the temperatures inside samples were measured with sensitive elements. The data were collected with a data acquisition/control unit and transmitted to a computer. After the tests the height and the water content of the freeze-and-thaw zone were measured. At least two parallel samples were studied.

The migration of $CaCl_2$ was studied by conducting water through specimens compacted with a gyratory compactor. The samples were set up in a tube with a specimen containing $CaCl_2$ on top and a specimen with water only on the bottom. Water was conducted from the top to the bottom of the sample. The amount of $CaCl_2$ was measured from the percolating water and the samples after the test. The test was complete when the amount of chloride in the percolating water reached the chloride content of tap water.

Results

The influence of $CaCl_2$ on basic properties of soils is due to its chemical properties, mentioned earlier. $CaCl_2$ has an effect on the finer particles of the aggregate, and the largest chloride contents are observed in the finer fractions (Figure 3). $CaCl_2$ also binds finer particles together. This phenomenon was observed in grain size distributions, in which the amount of finer particles decreased remarkably depending on the total amount of fine fractions (Figure 4). When materials with $CaCl_2$ were compacted, the amount of fine fractions showed only a slight decrease. This result indicates that the binding effect is permanent and cannot be broken mechanically. The binding effect can also be observed in the change of specific surface area: the values decrease remarkably (Table 1). The amount of water adsorbed on the surface increases when $CaCl_2$ is added. The results were nearly 10 times larger than the normal values. $CaCl_2$ changes the electrophoretic properties of fine fractions, as can be observed from results of zeta potential measurements in Table 1. The result indicates that the particles flocculate more easily. All these surface chemical properties correlate fairly well with $CaCl_2$ content of the sample (Figure 5).

The compaction studies at room temperature did not show results as remarkable as those mentioned in the literature. The maximum increase in the dry unit weight was 0.7 kN/m^3 (Figure 6). When the amount of the compaction effort was studied, the results were more encouraging. The compaction effort to achieve the same dry unit weight was an average 20 percent smaller when 1 percent $CaCl_2$ was added (Figure 7). The effect of

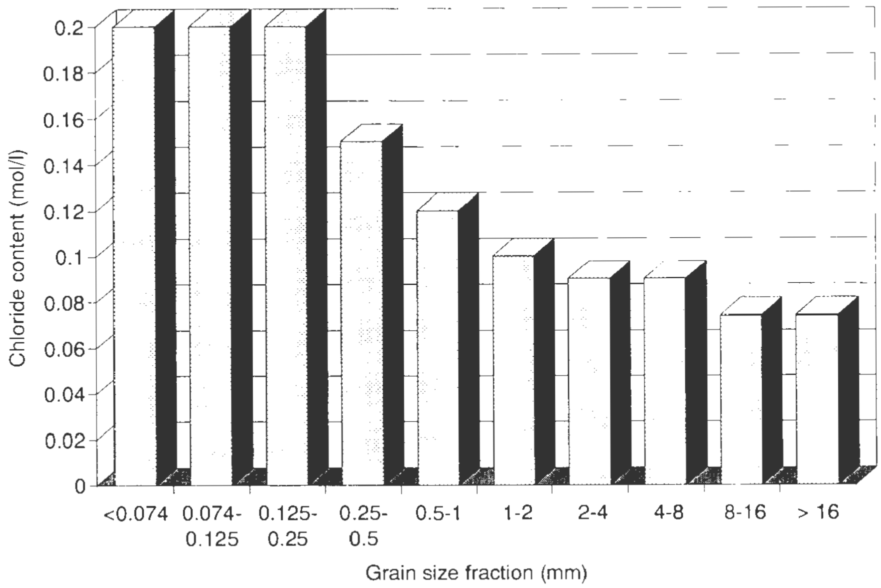


FIGURE 3 Chloride content of different grain size fractions of crushed rock aggregate.

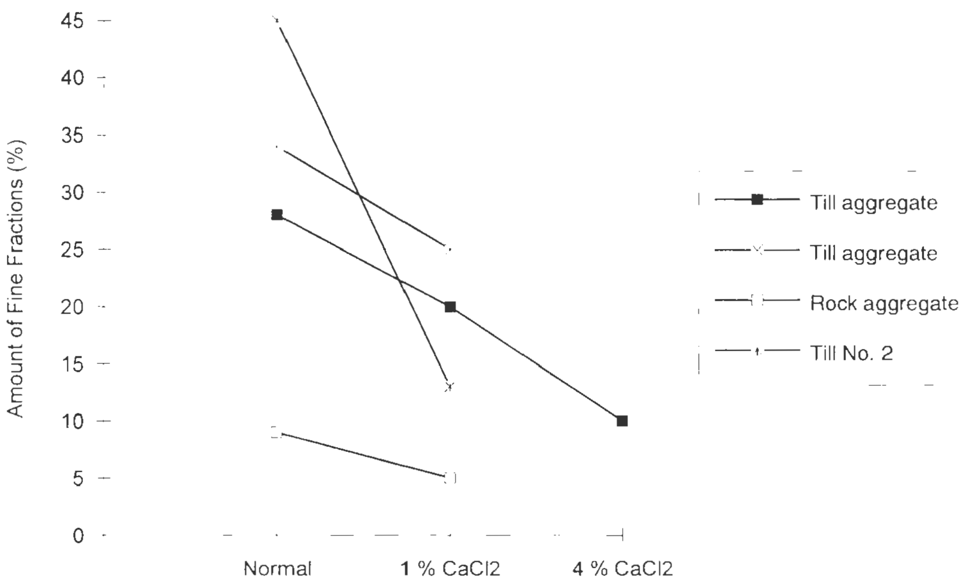


FIGURE 4 Change in amount of fine fractions in rock aggregate, till aggregate, and till after addition of 1 or 4 percent of liquid $CaCl_2$.

TABLE 1 Effect of $CaCl_2$ on Properties of Fine Fractions of Different Materials

Material	Water ads. index (%)	Sp. Surface Area (m2/g)	Zeta-potential (mV)
Till Aggregate	1.4	5.15	-22
-"- + 1 % CaCl2	12.7	0.89	-7
Rock Aggregate	2.1	2.5	-24.4
-"- + 1 % CaCl2	22.2	0.89	-6.6

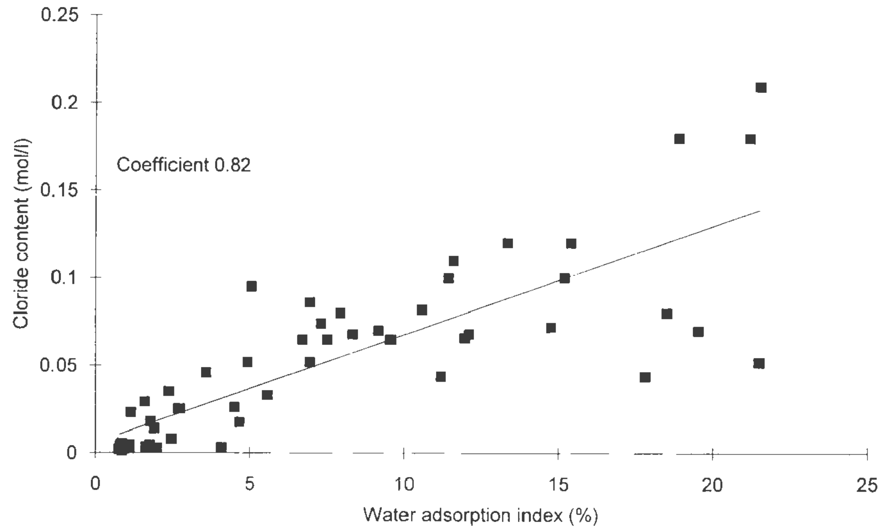


FIGURE 5 Correlation between chloride content and water adsorption index.

$CaCl_2$ on the compaction effort was strongest with materials containing larger amounts of fine fractions, such as glacial tills and crushed till aggregate. When the compacted specimens were stored in 60 percent relative humidity for a week, the water content of the specimen with $CaCl_2$ was 10 times larger than that of normal specimens.

Compaction of soil below 0°C temperatures always demands extra effort to achieve a firmly compacted layer and usually the dry unit weight is much lower than that obtained at room temperature. When the compacted material thaws, a structural collapse is often observed. By adding $CaCl_2$ the compaction becomes easier and the dry unit weight is nearly the same as it is at room temperature (Figure 8). The temperatures of the samples after compaction were still below 0°C.

The results of the frost heave tests indicated that $CaCl_2$ prevents frost heave. An 80 percent decrease in frost heave was even observed with frost-susceptible materials (Figures 9 and 10). Also the frost heaves of frost-resistant materials were smaller (on average 20 percent). Samples with $CaCl_2$ froze more slowly than those without it. When the temperature of normal sample was −4°C at a certain point after 24 h of freezing, in the sample with $CaCl_2$ the temperature was 0°C. After the test had proceeded for 48 h the temperatures were, respectively, −8°C and −4°C. The measurements of chloride content in the different parts of the sample

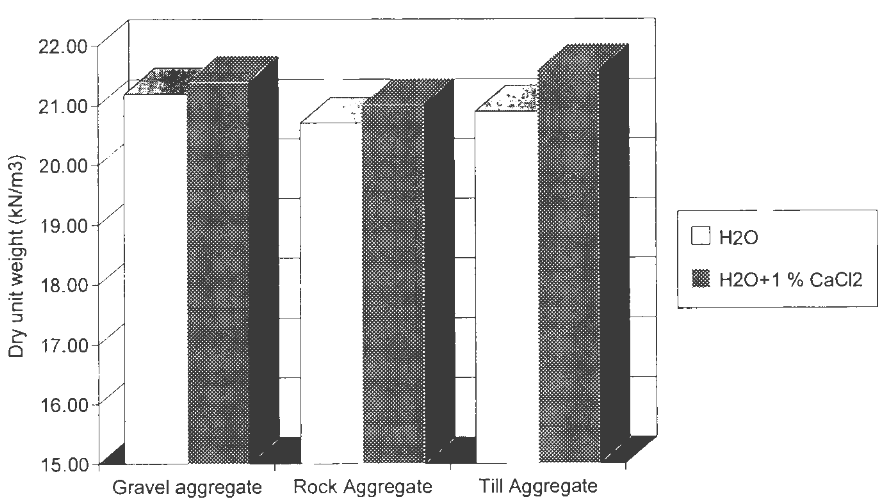


FIGURE 6 Effect of $CaCl_2$ on dry unit weight of different materials at room temperature.

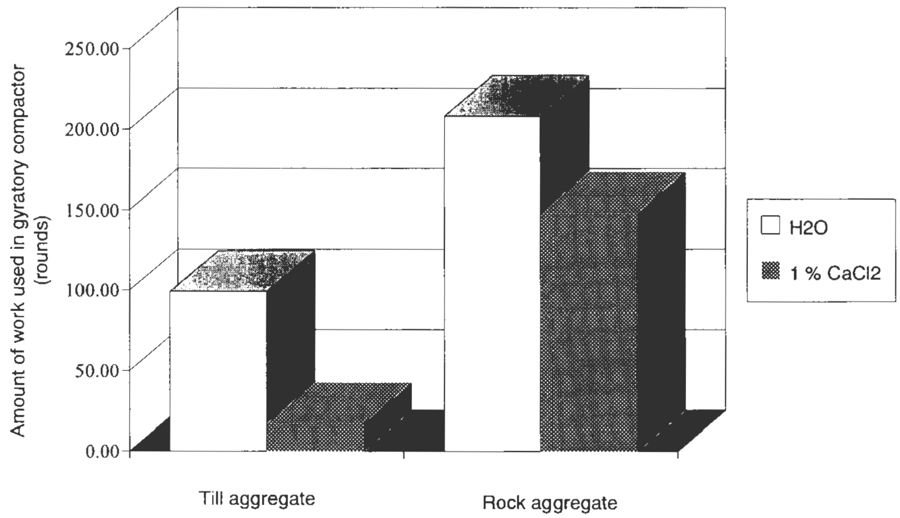


FIGURE 7 **Effect of $CaCl_2$ on compaction effort needed to achieve equal dry unit weight.**

showed that the content in the frozen part was nearly the same as before the test. In the thawed parts of the sample the chloride content showed a decrease because of the percolating water.

In the migration tests the results were not encouraging, although the test conditions were severe compared with field conditions. $CaCl_2$ disappeared from the specimens after 1 week. This was due to the solubility of $CaCl_2$. The best results were achieved with materials containing larger amounts of fine fractions. The amount of $CaCl_2$ in percolating water remained high for 2 days with till and till aggregate. When the gravel aggregate was tested, the amount of chloride decreased during the first day to a low level. This was partly due to the different velocities of water within different types of samples.

All the tests mentioned above were performed also for FDR materials. The results were not encouraging: $CaCl_2$ did not improve the compactability of the samples, and the permanence of salt in specimens was as poor as mentioned before. $CaCl_2$ did not increase the compression strength of FDR specimens; the compression strength, on the contrary, showed a small decrease. This was due to higher water content in samples with $CaCl_2$ than in those without it.

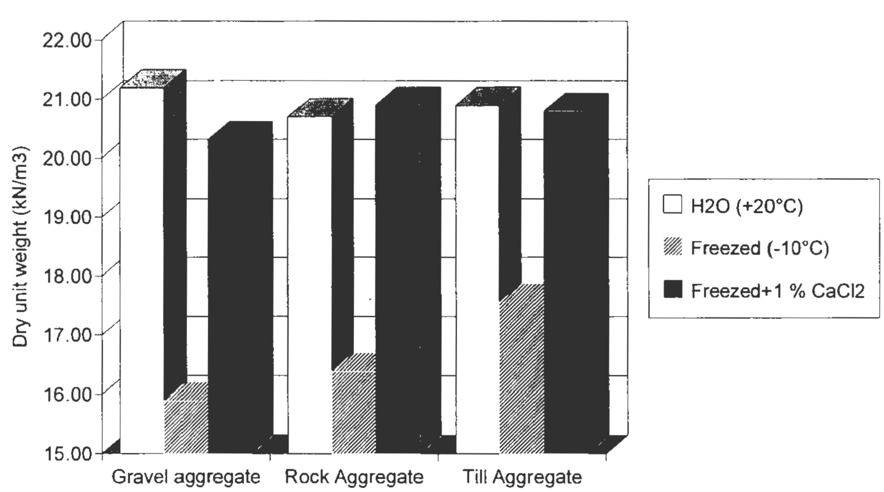


FIGURE 8 **Effect of $CaCl_2$ on dry unit weight of different materials below 0°C.**

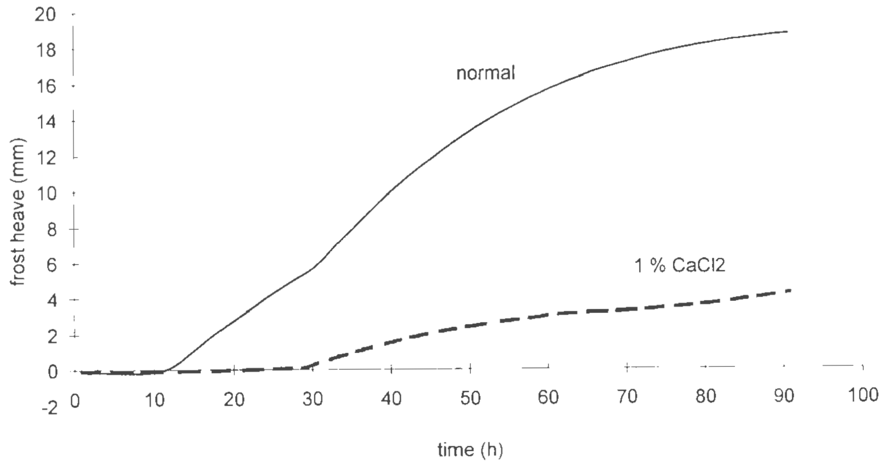


FIGURE 9 Effect of $CaCl_2$ on frost heave of frost-susceptible tills: weathered till No. 1.

Conclusions and Discussion

$CaCl_2$ binds the finer particles together, changing the basic properties of the fine fraction. The properties of whole aggregate material are changed by adding even only 1 percent of $CaCl_2$. This is due to the chemical properties of $CaCl_2$. Probably Ca^{2+} ions are adsorbing on the negatively charged small particles. A suitable method for evaluating the chloride content is to measure the changes in water adsorption index of fine fractions.

Calcium chloride does not increase the maximum dry unit weight in room temperature significantly but the compaction effort needed to achieve maximum dry unit weight decreases. The advantage of using $CaCl_2$ in compaction is its hygroscopicity. The material keeps its moisture for long periods; for example, in the summer the material need not be wetted often, and dust problems are avoided. During winter, frozen compaction becomes much easier if $CaCl_2$ is used and the collapses of structures in the spring are avoided. For example, when an excavation must be done in winter, the refilling and compaction with $CaCl_2$ could yield as compacted a layer as in summertime.

$CaCl_2$ has proven useful in preventing frost heave although there is no evidence that salt stays in the structure for long periods. The chemical nature of $CaCl_2$ makes the stability uncertain.

The migration of $CaCl_2$ is evident in contact with percolating water. However, in normal situations, there should be no running water in road construction. Ac-

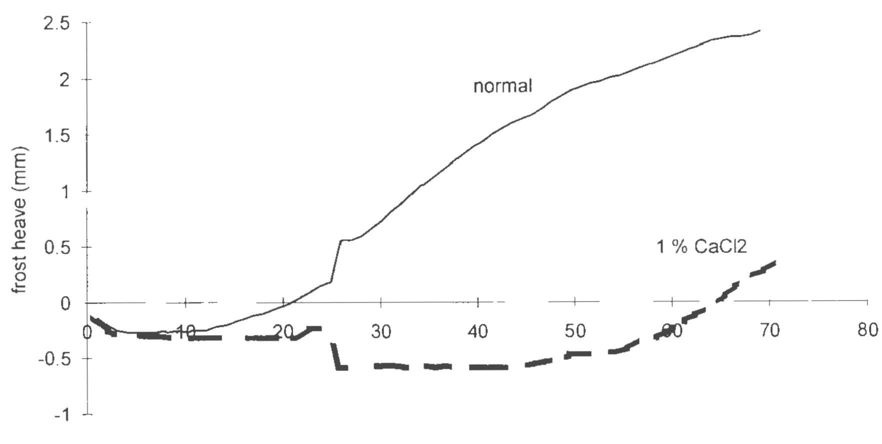


FIGURE 10 Effect of $CaCl_2$ on frost heave of frost-susceptible tills: glacial till No. 3.

cording to the present study no conclusions can be made on the stability of $CaCl_2$ in road construction.

The environmental effects of salt (mainly NaCl used in ice control) are widely discussed in Finland, and the chloride contents of groundwater are measured. Although the increase in the chloride contents of groundwater is only partly due to the NaCl used in ice control, there is evidence of its immediate effects. The effects of using $CaCl_2$ on road construction could also be harmful for groundwater. On the other hand, surface water flows into the groundwater aquifers through ditches and embankments. When salt is added in the base course the flow of water is prevented.

There are no unambiguous estimates on the effect of $CaCl_2$ on plants. The results of the literature review were contradictory. Also interesting is that $CaCl_2$ is used in greenhouse farming for moisture control. Road construction itself may be a greater environmental risk than the use of salt in either dust and ice control or in road construction. If $CaCl_2$ is added to the base or subbase layers during road construction, it might be more likely to stay in these layers than on the surface. The only way to prove this theory is to build a test road.

$CaCl_2$ is an effective additive during road construction to prevent dust, keep the material compactable, and decrease the needed compaction effort. $CaCl_2$ is also useful in wintertime construction. In all these applications the long-term stability is unimportant. In preventing frost heave, $CaCl_2$ is an efficient additive but its long-term stability is uncertain. The migration of salt is an environmental risk that should always be taken into account. During this research, the benefits of FDR with $CaCl_2$ could not be proven with laboratory research.

ACKNOWLEDGMENTS

The financial support of Kemira Chemicals Ltd. and the Finnish National Road Administration is gratefully acknowledged.

REFERENCES

1. Haas, W., D. Bernard, and T. Kaderabek. Using Additives to Improve Cold Weather Compaction. In *Transportation Research Record 560*. TRB, National Research Council, Washington, D.C., 1976, pp. 57–68.
2. Slesser, C. Movement of Calcium Chloride and Sodium Chloride in Soil. *HRB Proc.*, Vol. 23, 1943, pp. 460–468.
3. Slate, F. Use of Calcium Chloride in Subgrade Soils for Frost Prevention. *HRB Proc.*, Vol. 22, 1942, pp. 422–440.
4. Reckard, M. *Economic Aspects of High Speed Gravel Roads*. Final Report AKRD-83-20. FHWA; Alaska Department of Transportation, Juneau, 1982.
5. Shepard, J., J. Pickett, and M. Kienzle. Full-Depth Reclamation with Calcium Chloride. In *Transportation Research Record 1295*, TRB, National Research Council, Washington, D.C., 1991, pp. 87–94.
6. Reckard, M., and T. Kinney. *Effects of Salts on Road Embankment Stability Under Freezing and Thawing Conditions*. Final Report AKRD-87-09. FHWA; Alaska Department of Transportation and Public Facilities, Juneau, 1986.
7. *Presentation of 1992 FWD and Residual Chloride Testing Results, Chinguacousy Road, Caledon, Ontario*. Project G-02008-E/P. Trow Consulting Engineers Ltd., 1992.
8. Prior, G., and P. Berthouex. A Study of Salt Pollution of Soil by Highway Salting. In *Highway Research Record 79*, HRB, National Research Council, Washington, D.C., 1967, pp. 8–21.
9. Rich, A. Effects of Salt on Eastern Highway Trees. *American Nurseryman*, Vol. 11, No. 135, June 1972, pp. 36–42.
10. Nieminen, P., R. Salomaa, and R. Uusinoka. The Use of Specific Surface Area, Pore Volume, and Pore Area in Evaluating the Degree of Weathering of Fine Particles: An Example from Lauhavuori, Western Finland. *Fennia*, Vol. 163, 1985, pp. 373–377.
11. Kellomäki, A., and P. Nieminen. Adsorption of Water on the Fine Fractions of Finnish Tills. *Bulletin of Geological Society of Finland*, Vol. 58, Part 2, 1986, pp. 13–19.
12. Nieminen, P. Porosity and Related Properties of Frost Susceptible Tills. In *Proc., International Symposium of Frost in Geotechnical Engineering* (H. Rathmayer, ed.). Technical Research Centre of Finland, Espoo, 1989, pp. 557–564.

Field Studies on the Mechanical Behavior of Geosynthetic-Reinforced Unpaved Roads

Jacques Méry, *French Institute for Agricultural and Environmental Engineering Research*

Many French forest or agricultural engineers now use geosynthetics to protect base courses from clay contamination, but few of them design their roads taking into account the reinforcement effect. Several studies have been conducted since 1983 to evaluate this effect, using physical models and real structures. The usefulness of geosynthetic reinforcement for unpaved roads is analyzed, including practical and economical aspects.

Gravel roads are not as common in Europe as in large countries like the United States, Canada, or Australia. Nevertheless, they are widely used in agriculture and forestry as resource access roads because the low added value of such activities does not permit large investments. In Europe, France is particularly interested in unpaved roads because its rural areas are important. The length of the gravel-road network is unknown, but probably exceeds 200 000 km.

The design of such roads has rapidly included geotextiles to protect subbases from clay contamination, but more rarely has the reinforcement effect been considered. One reason for this is the absence of a design manual that explicitly indicates the expected thickness reduction. For temporary roads like improved subgrades, the Laboratoire Central des Ponts et Chaussées recommends a maximum thickness reduction from 13 to 19 percent (*1*), which is probably conservative in some cases since it takes into account the separation effect. The French guidelines on geotextiles in low-volume roads (*2*) gives some indication of the adequate geotextile properties but cannot be easily used to evaluate the reinforcement effect. Research has been carried out in the laboratory and in the field since 1983 to obtain useful data on the mechanical behavior of geotextile-reinforced unpaved rural and forestry roads.

One program was conducted in the laboratory (physical model) and in the field (on a rural road in the Massif Central), and another was held in timberland (on a public forest road in the Champagne-Ardenne region). The mechanical behavior was studied by simple field tests such as plate loading, deflexion of the Benkelman beam, and measurement of rutting due to accelerated traffic. Sections with different thicknesses and geosynthetics were compared. Practical and economical aspects, very important in this type of road, were also examined.

Rural Road in Massif Central

Presentation

The Massif Central is an old mountainous massif with a mean altitude of about 1000 m. The regrouping of lands in the Lozère Department required a new rural road network, and some experimental geotextile-reinforced sections were built in September 1984 with a grant from the Agriculture Department.

The design of the experimental roads was based on previous results obtained on physical models (a rigid box 2 m long and wide and 1.4 m deep in which different soil-fabric-aggregate systems are loaded by a plate), the details of which have been published elsewhere (*3*). They led to these conclusions:

- The reinforcing effect was important only for large plastic strains, corresponding to a rut depth of more than 5 cm;
- The anchoring of geotextiles did not reduce the deformability of the system, a result somewhat contradictory to other research work (*4*); and
- A two-layer geotextile structure (one geotextile between the subgrade and the base and one geotextile inside the base) has a greater reinforcing effect than a single-layer structure.

Experimental Sections

The experimental road comprised five 4-m-wide sections. The subgrade was a sandy clay (85 percent particles less than 80 μm, 50 percent particles less than 2 μm) with a California bearing ratio (CBR) of 3 percent at 15.5 kN/m^3 optimal density (Proctor normal French standard compaction). The aggregate cover was a local arenite 0/30 with 5 to 15 percent particles less than 80 μm. Its water content ranged from 4.9 to 8.5 percent during the construction (mean 6.4 percent), and ranged from 6.5 to 10 percent in May 1985 (mean 7.8 percent, close to the optimum for the CBR test).

One reference section (without a geotextile) was 15 m long and 30 cm thick. Three sections 50 m long and 20 cm thick had a two-layer reinforcement of a woven geotextile (main transverse properties according to French standards: 525 g/m^2 mass per unit area, 75 kN/m tensile strength, 18 percent elongation at failure), a spunbonded nonwoven geotextile (280 g/m^2 mass per unit area, 18 kN/m tensile strength, 47 percent elongation at failure), and a needlepunched grid-reinforced nonwoven geotextile (this experimental fabric has no published properties). The last section, 120 m long and 20 cm thick, had only one layer of the spunbonded nonwoven geotextile.

Tests in 1985

In early spring 1985, a first test was carried out on all the sections except the single-layer reinforced layer. The real thicknesses were found to be somewhat different from the design values: 16 to 18 cm for the woven geotextile, 21 to 24 cm for the spunbonded nonwoven geotextile, and 32 to 36 cm for the needlepunched nonwoven geotextile.

The accelerated traffic (11 loading cycles by a 130-kN rear axle) and elastic modulus (about 35 MPa) derived from a 30-cm-diameter plate loading test were similar on all the sections. The two-layer structures showed a vertical displacement less than 5 cm on the surface (rut depth) and less than 4 cm for the upper geotextiles, whereas the reference section showed rut depths from 10 to 17 cm.

Plate tests at high stress (0.5 to 0.8 MPa) also exhibited better behavior for the reinforced section. The exception was the section with the woven geotextile, whose subgrade had the highest water content—30 percent instead of 20 to 24 percent elsewhere—and whose thickness was the lowest.

The deformation pattern was similar for upper and lower geotextile in each structure. Detailed results were published elsewhere (*3*)

Tests in 1987

In early spring 1987, investigations confirmed some unevenness in pavement thickness and subgrade strength. It was difficult to classify the reinforcement ability of the different geosynthetics without making detailed measurements. At the same point on each section, pavement thickness tests, plate loading tests, and trench openings were made after accelerated traffic.

The high-stress plate loading seemed to better distinguish the nonwoven spunbonded and the woven-geotextile-reinforced sections: for a 4-mm plate displacement, the vertical stress is 0.1 MPa for the reference section (>41 cm thick), 0.4 MPa for the woven-geotextile-reinforced section (18 cm thick), 0.5 MPa for the spunbonded nonwoven-geotextile-reinforced section (22.5 cm thick), and 0.6 MPa for the needlepunched nonwoven-geotextile-reinforced section (40 cm thick).

The accelerated traffic tests (16 loading cycles by a 210-kN rear tandem axle) showed lower ruts than in 1985: 1 cm on the reference section and the needlepunched nonwoven-geotextile-reinforced section, 2 cm on the spunbonded nonwoven-geotextile-reinforced section, and between 2 and 3 cm on the woven-geotextile-reinforced section.

According to the plate tests and elasticity theory, the subgrade would have a CBR of about 6, but the reference section would have a CBR of about 10. On the single-layer section (12.5 cm thick, computed CBR = 5), the accelerated traffic (12 times loading cycles by the same axle) generated 0.5-cm-deep ruts while deflections were more than 500/100 mm.

Overview of Results

Taking into account subgrade CBR and aggregate thickness, plate loading tests and accelerated traffic tests generally showed better behavior for the woven- and spunbonded nonwoven-geotextile-reinforced sections.

The reference section (40 cm thick and CBR = 10) and the single-layer geotextile-reinforced section (12.5 cm thick and CBR = 5) did not show significant differences in rut depth for similar loading cycles. According to elastic theory or empirical findings (5), the influence of pavement thicknesses is relatively greater than the influence of subgrade modulus on the mechanical behavior of roads; thus the geotextile would reduce the rut depth.

The double- and single-layer geotextile-reinforced sections did not show significant differences in rut depth for similar loading cycles either, so the two-layer structure is not so efficient (for rutting) as may be expected from the laboratory physical model.

The rutting tests were made for very few loading cycles. For more loading cycles, significant differences in rut depth might, however, appear.

Forest Road in Champagne-Ardenne Region

Presentation

Many timberlands in northern France lie on clayey soils, and harvesting often occurs during the cold and wet season. Geotechnical problems are then commonly encountered by foresters, especially in road engineering. Accordingly, the National Forest Office, Champagne-Ardenne Region, has funded research in the field of geosynthetic-reinforced roads since 1989. The experiment related here began in 1992 near Troyes.

Previous experiments (6) showed that aggregate thickness and subgrade should be as homogeneous as possible. The continuous checks during the work (not part of common practice because of limited human resources) and the homogeneity of the site allowed these conditions to be fulfilled. Single-layer geosynthetic-reinforced sections were designed to cost approximately 25 percent less than the normal regional cost. It is thus more an economic comparison than a purely scientific one. Two-layer geosynthetic-reinforced sections were not tested since the foresters would not accept an apparent overcost of \$3/m^2 or more for materials.

Experimental Sections

Eight sections were built by combining three geosynthetics and different aggregate thicknesses. The length was short (315 m) to permit easily detailed investigations.

The subgrade undrained cohesion evaluated by an unconsolidated, undrained triaxial test was about 60 kPa (with a 34.6 percent water content). In the field, vane shear tests indicated values from 80 to 95 kPa (with a 36.5 percent mean water content), which is far more regular than in the first experiment. Taking into account the plasticity index of the subgrade (35 percent), the real undrained cohesion may vary from 70 to 85 kPa. No water content measurements were made on the limestone crushed aggregate, but this seemed to be too dry for an optimal compaction.

There was one reference section 40 cm thick (called R40) with a spunbonded nonwoven geotextile (290 g/m^2 mass per unit area, 21 kN/m tensile strength, 46 percent elongation at failure), which represented the normal regional design. Two sections were 30 cm thick, with the same geotextile and with a woven one, particularly resistant (330 g/m^2 mass per unit area, 55 kN/m tensile strength, 10 percent elongation at failure). These were respectively called S30 and W30. Three sections were 20 cm thick with the same two geotextiles and with a geogrid (200 g/m^2 mass per unit area, 20.5 kN/m tensile strength, 10 percent elongation at failure), respectively called S20, W20, and G20. Two 15-cm-thick sections, one with the same geogrid and one without a geogrid (reference section), were respectively called G15 and R15.

What was new in the second experiment was the testing of a common geogrid and the easy comparison between each section due to even subgrade strength and aggregate thickness.

Tests in 1993

The same measurements as in the first experiment were done in early spring 1993. The results, and the undrained cohesion corresponding to each section, are reported in Table 1.

The deflection test showed that the type of geosynthetic had less influence than aggregate thickness, even for the geogrid. S30 was stiffer than R40, but no measurements of the subgrade cohesion were taken here. Some variations on aggregate quality might explain this result on such thick sections: aggregate grade is more crude on R40 (0/100 instead of 0/60).

The reinforced 15-cm-thick section was surprisingly more deformable than the nonreinforced one, according to plate tests. The slightly lower undrained cohesion of the reinforced section (80 MPa in comparison with 85 MPa) might explain this result on such a thin section. The maximum measurable 2-cm plate-sinking is not sufficient to stretch the geogrid, but it seems difficult to invoke some slippage of the cover on the geogrid at low plate-sinking as could be the case for geotextiles, especially woven ones, considering their frictional properties.

TABLE 1 Results of Tests Carried Out in 1993

	Section							
	R40	S30	W30	G20	S20	W20	G15	R15
Undrained cohesion (kPa)	—[a]	—	—	—	85	70	80	85
Deflection by 190-kN axle (mm)	6.8	4.7	8.7	12.3	11.6	13.8	15.1	
Plate-sinking at 0.35 MPa (mm)	—	—	—	8.6	10.4	10.0	19.2	14.3
Rut depth (cm)								
At 5 cycles	—	—	—	2	3	3	5	5
At 10 cycles	—	—	—	5	5	10	10	10
At 30 cycles	—	—	—	8	12	19	17	23

[a]No measurements made.

For the same 20-cm thickness, G20 behaved the best according to the plate test. Undrained cohesion below G20 was unfortunately not measured during the tests. It is assumed to be slightly higher than that below W20 from laboratory CBR tests issued before the construction of the road.

The accelerated traffic test (30 loading cycles by a 130-kN rear axle), carried out on the 15- and 20-cm-thick section only, showed limited rut depths until five loadings, then higher ones.

The shape of the deformed geosynthetics after 30 truck loadings is reported in Figure 1 for the 15- and 20-cm-thick sections except G20. By comparison with R15, G15 showed the well-known influence of fabric on the deformation mechanism (larger soil mass involved in the plastic deformations). The spunbonded

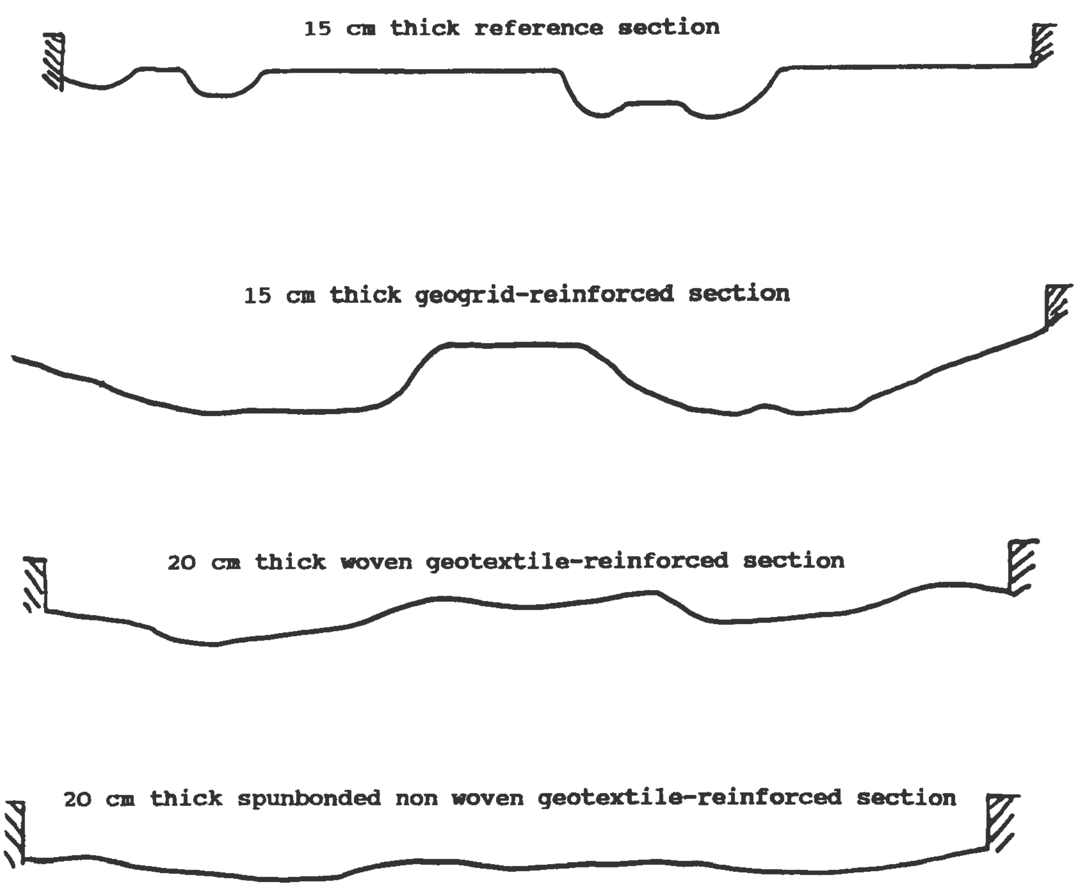


FIGURE 1 Shape of deformed geosynthetics after 30 truck loadings (scale 1/20).

nonwoven geotextile was less rutted than the woven one. These results are consistent with rut depth measurements. The spreading angle after repeated loadings, a classical parameter in several design methods, was evaluated by the distance between the inflection points, whose location is not always as accurate as desired (see Figure 1) according to Riondy (*4*). The highest value, 45 degrees, is found in G15, the middle value of 35 degrees in S20, and the lowest value of 28 degrees in W20. Other measurements are desirable to confirm that these angles do depend on the fabrics, which was verified in small-scale models (*4*). R15 showed an angle close to 25 degrees, not far from the theoretical angle of 45-phi/2, phi being the friction angle of the crushed limestone, probably close to 35 degrees.

Note at least that the deformed shapes are roughly symmetrical toward the center of the road, but not toward the wheels (higher curvature inside, lower curvature outside). This fact is not always accounted for in physical or analytical models, although it seems foreseeable that the soil-fabric-aggregate system has asymmetrical behavior toward the wheels (the transverse profile geometry is not symmetrical toward them).

Overview of Results

Taking into account the subgrade cohesion, it can be argued from plate tests on the 20-cm-thick sections that (*a*) the geogrid strengthens the road more than the spunbonded nonwoven geotextile, and (*b*) the woven geotextile strengthens the road more than the spunbounded nonwoven geotextile.

Considering the rutting tests, S20 shows from 30 to 50 percent less degradation than W20 while the variation of subgrade cohesion is less than 20 percent. Thus the spunbonded nonwoven geotextile would strengthen the road more than the woven geotextile.

It appears that the different tests do not always give the same geosynthetic "rank" for a given section. The deflection test is generally not sufficient to evaluate geosynthetic reinforcement because the structures are not deformed enough. From a practical point of view, the accelerated traffic test is probably the most relevant since loading is very close to reality and takes into account the dynamic and repetitive effect of traffic. Moreover, it allows statistical analyses, which are more difficult to deduce from plate tests.

According to the accelerated traffic test, W20 does not perform as well as G20 or even as well as S20, although the woven geotextile has the highest strength and the nonwoven has the lowest stiffness. Other properties such as friction or even flexibility may play a role in general mechanical behavior, particularly in the rut formation. The reinforcement effect for small rut depth, studied more in recent years (*7*), could invoke these properties instead of the classical geotextile properties used to compute membrane effects (tensile strength and modulus).

Practical and Economical Approach

Practical Approach

The research presented here is devoted to a better use of geosynthetics in road engineering to reduce the investment and maintenance costs of rural and forestry unpaved roads. The aim is not to develop purely theoretical models. Therefore, some care must be taken before modeling reinforcement effects: the modeling hypotheses must be close to what occurs in the field. Some remarks are necessary here.

First, the membrane effect is negligible unless rut depth exceeds 5 to 10 cm (*3,4*), which is not desirable for permanent rural and forestry uses. Graders and rollers are not continuously available to quickly offset deep ruts as is the case for construction traffic roads or improved subgrades. Moreover, recreational use could require good serviceability, hence limited rut depth. Theories based on the membrane effect have been well developed because they are readily adapted to computations, but they do not entirely meet the real needs of unpaved concerning rut depth.

This membrane effect depends on the frictional stress outside the axle, which depends itself on the thickness and the width of the aggregate cover, and to a certain extent on fabric anchorage. These are particularly low on rural and forestry roads, and anchoring is not common practice.

The stress applied to geosynthetics during construction may be higher than that applied in service. The second experiment suggested this, although no measurements were made. Localized rheological contrasts (stones, stumps, roots, soft spots) can stretch or puncture geosynthetics under construction traffic. For that reason, foresters in the Champagne-Ardenne region do not use geosynthetic whose mass per unit area is less than 200 g/m^2. Geogrids have been revealed to be very sensitive to this kind of stress. In many situations (only one access, very narrow subgrade) it is difficult for construction traffic not to work directly on the geosynthetics, so this problem has to be accounted for.

The particularly narrow width of rural and forestry roads theoretically requires short rolls, which are not always available since the main market for geosynthetics is highways. The rolls width is not always optimized for rural roads. Anchoring the extra-width geotextile in the aggregate cover does not provide more reinforcement (*3*). The roll may be cut at the desired width, which is not recommended for geotextiles, especially woven ones. This must be done for geogrids because of their higher flexural stiffness, but the cut is much easier than for geotextiles.

Last but not least, the handling of geogrid rolls is much easier than reinforcement geotextile rolls, except perhaps some woven geotextile with sufficient tensile strength and low mass per unit area, since the weight per unit area of geogrids is particularly low.

Economical Approach

Geosynthetics will not be developed for reinforced unpaved road unless their cost is lower than the cost of the aggregate saved by the reinforcement. The results obviously depend on the cost difference between aggregate and fabrics and must be adapted to each case. For instance, the National Forest Office found that common geotextiles were worthwhile but not geogrids in the Ardenne Department.

Other advantages should be taken into account, such as these:

- Less damage to the access road network, helping maintain cordial relationships with forest managers, who may otherwise forbid haul traffic, as occurs in some French timberlands;
- Less likelihood of encountering bad weather conditions because of the shorter duration of the work (these advantages do not easily lend themselves to economical computations); and
- Lower maintenance cost.

Research programs on geosynthetic-reinforced unpaved roads that take into account the influence of fabrics on maintenance cost are welcome.

Conclusion

The results of the two experimental programs did not clearly show the key geosynthetic properties for reinforcement purposes, nor the expected aggregate thickness reduction. For heavy traffic, it can be argued that building roads with an aggregate thickness of less than 20 cm is not recommended, whatever the geosynthetic. The thickness reduction range due to the geosynthetic reinforcement is usually evaluated to be from 10 to 40 percent (even more according to some fabric producers). The experiments presented here confirm this.

It seems that all fabrics do not generate the same mechanical behavior. The differences do not directly depend on geosynthetic modulus, a result already found by others when there is no anchoring (*4*), or when the subgrade is compressible (*8*).

The results and reflections presented here show the state of the art in designing geosynthetic-reinforced unpaved rural and forestry roads in France. Design thickness and geosynthetic survivabiliy specifications seem less crude in the United States (*9*) than in France (*2*). Research work should now be disseminated more widely, with refinements carried out to optimize geosynthetic choices for rural and forestry unpaved roads.

ACKNOWLEDGMENTS

The author wishes to thank M. Jamet for the field measurement work, the Agricultural Department and the National Forest Office for providing the experimental sites and the research funds, and the fabric producers for providing the geosynthetics.

REFERENCES

1. *Réalisation des remblais et des couches de forme.* Guide Technique SETRA-LCPC. Paris, France, 1992.
2. *Recommandations pour l'emploi des géotextiles dans les voies de circulation provisoire, les voies à faible trafic et les couches de forme.* Comité Français des Géotextiles et Géomembranes, Paris, France, 1981.
3. de Garidel, R., and E. Javor. Mechanical Reinforcement of Low-Volume Roads by Geotextiles. *Proc., 3rd International Conference on Geotextiles,* Vienna, Austria, Vol. 1, 1986, pp. 147–152.
4. Riondy, G. *Géotextiles et comportement à la fatigue des chaussées non revêtues.* Ph.D. thesis (in French). University of Grenoble, France, 1985.
5. Webster, S. L., and S. J. Alford. *Investigation of Construction Concepts for Pavements Across Soft Ground.* Technical Report S-78-6. U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., 1978.
6. Méry, J., and R. de Garidel. Geosynthetic Reinforced Forest Roads. *Recontres 93, Comité Français des Géotextiles et Géomembranes,* Joué les Tours, Vol. 1, 1993, pp. 97–106.
7. Houlsby, G. T., and R. A. Jewell. Design of Reinforced Unpaved Roads for Small Rut Depths. *Proc., 4th International Conference on Geotextiles, Geomembranes and Related Products,* La Haye, 1990, pp. 171–176.
8. Douglas, R. A., R. B. Bessey, and R. P. Small. The Use of Geotextiles in Forest Road Construction. *Proc., 2nd Canadian Symposium on Geotextiles and Geomembranes,* Edmonton, Alberta, Canada, 1985, pp. 89–96.
9. *Aggregate Surfacing Design Guide.* Final Report. Forest Service, U.S. Department of Agriculture, 1990.

Launched Soil Nails: New Method for Rapid Low-Impact Slope Repairs

John E. Steward, *USDA Forest Service, Missoula, Montana*
J. Mauricio Ribera, *USDA Forest Service, Corvallis, Oregon*

A variety of methods have been used during the last 20 years to reinforce soils. One of these is soil nailing. Most often, soil nails are installed by inserting steel rods in drilled holes, then grouting them in place. Sometimes the nails are inserted using percussion methods. These methods generally require excavation of a working bench in order for the construction equipment to work below the slope being nailed. These methods are not suitable for repairing small slips of road fills and embankments where access is limited. Launched soil nailing, a new technique developed in the United Kingdom by Soil Nailing, Ltd., allows nails to be inserted into the slope using a launcher attached to the end of an excavator boom. With this method the nails can be installed into slopes up to 8 to 11 m (26 to 36 ft) above or below the road surface without excavation or ground disturbance. The launcher uses high-pressure compressed air to install the nail. The depth of penetration depends both on the compressed air pressure and on the in situ material. At a reproduction rate of 15 nails per hour, this method is rapid, yielding production results not experienced using conventional methods. In July and August 1992, the USDA Forest Service sponsored a demonstration project for launched soil nailing in the western United States. The project successfully demonstrated the feasibility of using launched soil nails to stabilize failing road slopes. Small slope failures [no deeper than about 4.5 m (15 ft)] can be stabilized for about $150/m^2 ($14/ft^2) of slope face. Low retaining walls and excavate-and-replace methods typically cost $161 to $645+/m^2 ($15 to $60+/ft^2) of face area. Equipment mobility, rapid placement, minimum site disturbance, and low costs indicate a strong future for launched soil nails for the repair of the road infrastructure.

In July and August 1992, the USDA Forest Service and Soil Nailing Limited from the United Kingdom sponsored a demonstration project for the launched soil nail method in the western United States. The demonstration involved installation of launched soil nails at eight sites in four states and three Forest Service regions. Demonstrations included soil nailing of road shoulders, retaining walls, a cut bank, and a sand bank. Financial assistance was provided by the FHWA Coordinated Technology Implementation Program (CTIP). Technical assistance was provided by the Washington and Colorado departments of transportation, and seven national forests that participated in the demonstration project.

The project was developed to demonstrate the use of launched soil nails to repair and reinforce unstable cut bank and embankment slopes. The demonstrations provided an opportunity for engineers, maintenance personnel, and contractors to view and explore the potential for using launched soil nails.

A video, *Application Guide for Launched Soil Nails* (*1*), and the project report (2) are products of the demonstration project. Demonstration site experiences, participant interviews and questionnaires, and a simplified wedge analysis for soil nailing provide the basis for this paper.

Demonstration Project Results

The soil nail launcher successfully installed galvanized steel nails with diameters of 38 mm (1.5 in.) (3) and lengths of 5.4 m (18 ft) into a wide variety of materials. Launcher air pressures of between 4.1 and 17.2 kPa (600 to 2500 psi) resulted in nail tip penetrations of 1.5 to 5.4 m (5 to 18+ ft).

A production rate of 15 nails per hour was achieved by a three-person work crew (launcher operator and two helpers).

Participants in the demonstrations indicated the following:

- For small road failures, equipment can be moved in, the failure repaired, and the equipment moved out in less than 1 day.
- The method has high potential for slope reinforcement, especially on road shoulders and backslopes.
- The method has medium potential for retaining wall reinforcement, horizontal drains, and anchor insertion.
- Either tracked or rubber-tired excavators are suitable, although tracked excavators may be more versatile.
- Using a self-propelled, rubber-tired excavator for road shoulder repairs could eliminate the excavator hauling unit.
- Minimum ground disturbance and mobility are important features of the technology.
- Potential limitations of the technology include the length of nails (limited to smaller slides), penetration in cobbly soil, and maintaining precise control over depth of nail penetration.
- Design concerns include nail pullout resistance; need for practical design guidance; need for more experience with the technology, including case histories; and corrosion of permanent installations.
- Facing systems would be appropriate for temporary walls and for very shallow slides or erodible soils.

Several actions have been initiated as a result of these demonstrations:

- Design charts for stabilizing road shoulders with launched soil nails were developed (see Figure 1).
- A longer-term (1- to 2-year) demonstration project is being planned to gain experience with completed projects and to develop case histories.
- More exposure is planned to increase awareness of this technology by engineers, contractors, and maintenance personnel.

Soil Nailing Using Launched Nails

Soil nailing is a reinforcement technique that inserts long steel rods into an unstable or potentially unstable existing soil mass. Soil nails installed into the soil act to reinforce the soil mass by transferring tensile and shear resistance of the nail to the soil. The nails maintain the restraint force because they are anchored beyond the slip surface. Figure 2 shows how these forces act to retain a small soil slip.

During the past 20 years, a variety of methods have been used to install soil nails. Most often they are inserted into drilled holes and then grouted in place. Sometimes they are driven into the soil using percussion methods. These methods generally require the excavation of a working bench (Figure 3) for the equipment and are not suitable for repair of small slips in road fills and embankments where access is limited. These methods also require the removal and replacement of soils, often resulting in large areas of disturbed and raw ground. Because the soils are moved twice and the drilling is slow, costs are generally as high as those for other retaining structures. Environmentally, the disturbed and raw ground requires time to heal, stabilize, and provide desirable ground cover.

Launched soil nailing, also called ballistic soil nailing, is a new technique developed in the United Kingdom. Soil nails are installed by means of a launcher mounted on a hydraulic excavator (Figures 4 and 5). The launcher uses high-pressure air acting upon a collet (plastic collar) attached at the tip (front end) of the nail (Figure 6). Compressed air suddenly released against the collet forces the collet and nail through the launcher barrel, much like a dart through a blow gun (Figure 3). Launched soil nails are installed rapidly with little soil disturbance.

The nails are launched at speeds of over 320 km/hr (200 mph) and at pressures approaching 17.2 MPa (2,500 psi). The collet breaks away as the nail enters the soil. As the launched nail passes into the soil, the ground around the nail is displaced by compression at the tip. This forms an annulus of compression as shown in Figure 7(*f*), reducing soil-drag on the nail and damage to the galvanized coating. Depth of nail penetration is normally controlled by air pressure and ground resistance. Optionally, the nail penetration can be arrested by fitting the end of the nail with a tapered screw-on coupling as shown in Figure 7(*e*). During launching, the force (air pressure) acts upon the tip of the nail, thus

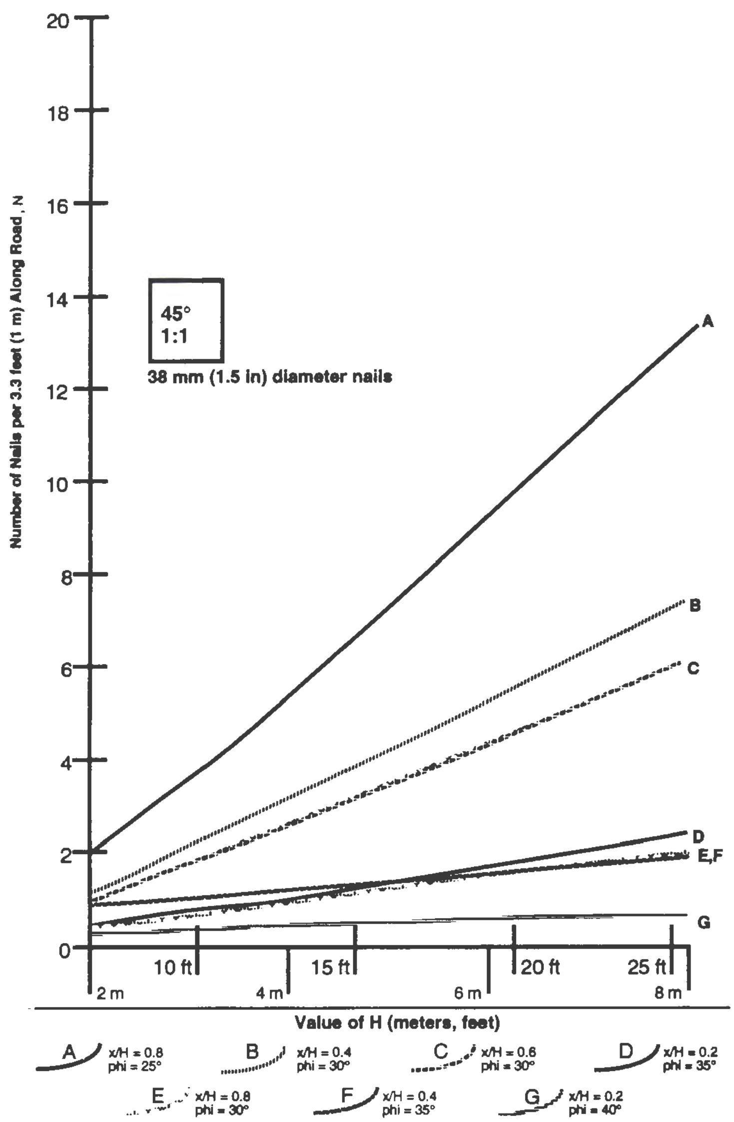


FIGURE 1 **Number of nails required to stabilize road shoulder for 1:1 (45-degree) slope.**

placing the nail temporarily in tension and preventing it from buckling.

The launcher typically launches plain or galvanized steel nails up to 38 mm (1.5 in.) in diameter and up to 6 m (20 ft) long (the 1992 demonstration used shorter nails—5.5 m (18 ft) long due to the length of the shipping container). The nail should be oriented normal to the potential slip plane to act primarily in shear and bending, with the tension being induced by movement.

Depending on the length of the boom, the launcher can be positioned 1.5 to 11 m (5 to 35 ft) above or below the excavator's platform. The launcher is attached to the boom by an articulated knuckle (Figure 4) that allows tilting of the launcher at almost any desired angle. Excavation for a working bench is usually not needed for road repairs using the launcher. The soil nail launcher has been used in the United Kingdom to successfully install nails in a variety of soil and slope conditions, primarily for reinforcement of road and railroad embankments and to strengthen retaining walls. Before this demonstration, the equipment had not been used in the United States.

Need for Launched Soil Nails

Roads constructed on steep slopes are susceptible to sliding and shoulder cracking (Figure 8). These cracks

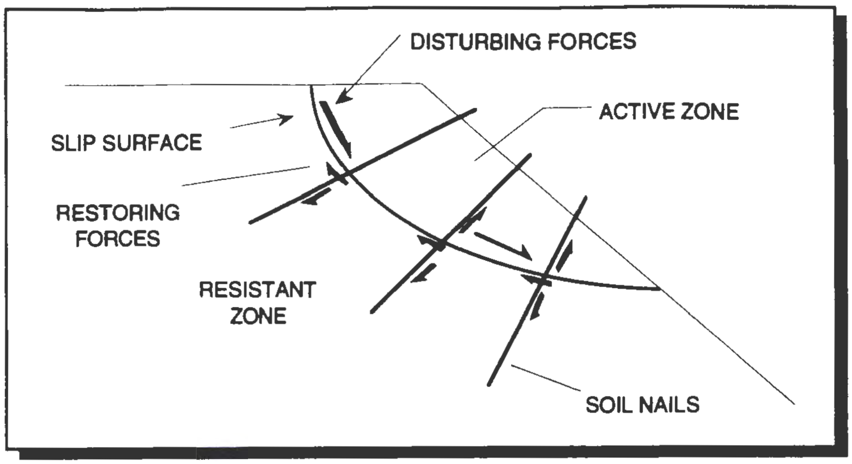


FIGURE 2 Forces acting on a road slope failure.

allow water from rain and snow melt to enter, adding excess moisture and water pressure directly to the slide mass. These areas are periodically filled and patched to smooth the road, adding weight to the sliding mass and further decreasing stability. Such fill failures are costly to repair, impair safe travel, and can cause extensive damage to the surrounding land and streams. Obviously, permanent repair methods are preferred over the annual crack-filling and patching of these unstable areas.

Launched soil nails offer a rapid economical alternative to recurring maintenance or other reconstruction solutions. Often several small fill failures can be fixed in one day without excavation. The launcher can be moved with ease between trees and shrubs, resulting in little or no vegetation removal and little need for environmental or visual mitigation (Figure 5).

The soil nail launcher, which weighs about 1250 kg (2,750 lb) mounted on a standard hydraulic excavator, is highly mobile and can respond quickly. Small slides can be quickly stabilized before they progress into larger slides. This quick response prevents more expensive repairs and further environmental damage.

Designing with Launched Soil Nails

A number of methods can be used to account for the reinforcement benefit to the slope using launched soil nails. Soil Nailing Limited developed a design method

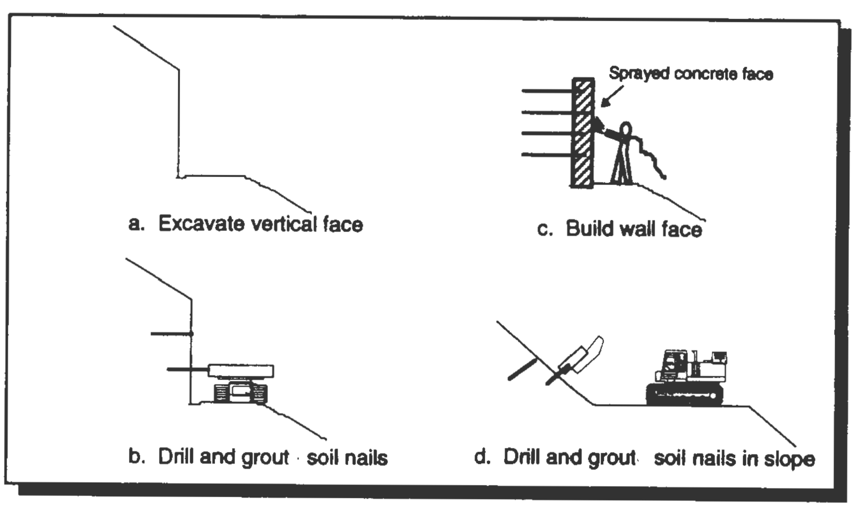


FIGURE 3 Conventional soil nailing.

FIGURE 4 **Closeup of launcher, with numbers indicating (*1*) nail guide, (*2*) air chamber and valve, (*3*) barrel, (*4*) noise and debris shroud, (*5*) spring-loaded safety switch, (*6*) articulated knuckle, and (*7*) excavator boom.**

FIGURE 5 **Nails installed in road shoulder without disturbing vegetation. Nails were cut off at ground surface. Note pavement displacement in right foreground (*1*).**

using a simplified wedge analysis (Figure 9) (*1*). The soil nails impart both tensile and shear resistance from the nail to the soil as shown in Figures 10(*a*) and 10(*b*).

The 1992 demonstration project provided a qualitative demonstration of the equipment capability. Sites were not designed to test the stabilization of moving slopes. However, areas of known movement were selected for most demonstration sites to judge potential performance.

As a result of the field demonstrations and work with technical advisors, the simplified wedge design methodology was developed to aid in selecting nail spacing to stabilize small road shoulder slides on low-volume roads. Typically these slides may require 15 to 50 nails for stabilization at a cost of $2,000 to $6,000 per site. Geotechnical drilling can cost $3,000 to $10,000 and is usually not warranted for these slides. The design method assumes that a site evaluation has been performed by experienced geotechnical personnel, usually without exploratory drilling.

Where a slope has failed or is near failure, it can be said that the soil profile has a factor of safety equal to 1. During the project, design charts were developed to calculate the number of nails needed per meter length of road for embankment slopes of 1:2 (26°), 1:1.5 (33°), and 1:1 (45°), respectively, to increase the slope factor of safety to 1.1. Like mechanically stabilized embankments (MSE), factors of safety are applied to the reinforcement. The design charts incorporate a material factor of safety (f_m) of 2 against pull-out and shear or tensile failure of reinforcement.

Additional assumptions for the design charts include the following:

- The slope has been in place for a number of years and can be represented by the consolidated-undrained condition during slope movement.
- The soil strength can be represented by an effective cohesion of zero ($C' = 0$) and an apparent angle of internal friction of ϕ estimated from site failure geometry, soil classification, and seepage conditions.
- Groundwater and seepage pressures are either minimal or controlled by installed drainage.
- Nails are installed nearly normal to the slide plane.
- The depth through the active zone into the resistant zone and in the active zone is at least 1 m (3.3 ft) to develop nail resistance.

FIGURE 6 **Soil nail ready for insertion loading into launcher. Compressed air is introduced between locking washer (*1*) and collet (*2*). The collet separates from nail in noise and debris shroud.**

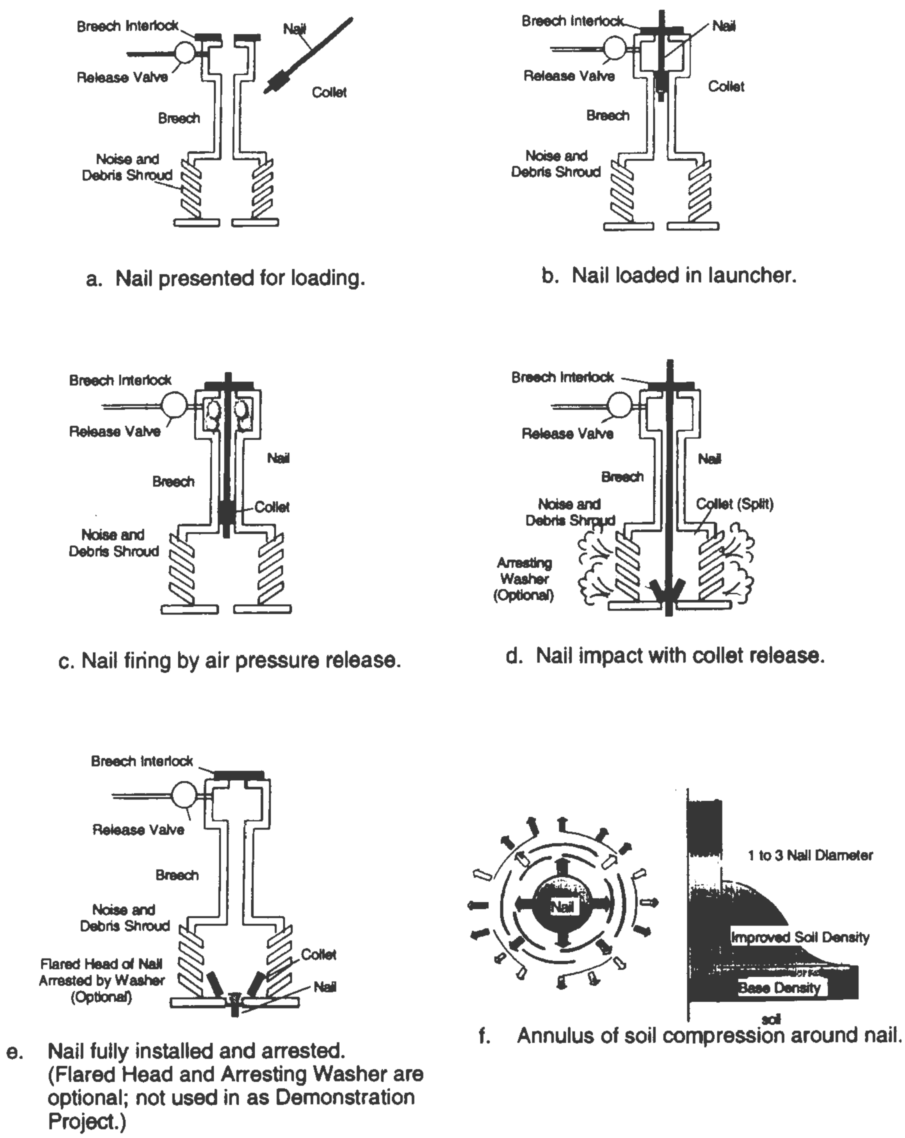


FIGURE 7 Soil nail launching sequence.

- The top row of nails is placed about 1 m from the road shoulder, the bottom row of nails is no closer than 1 m above the toe of the slide, and the remaining nails are evenly distributed throughout the slide mass.

Figure 1 shows the design chart for a 1:1 (45-degree) slope. The full design method and design charts for 1:2 (26-degree) and 1:1.5 (33-degree) slopes are contained in the application guide (*1*).

To ensure full penetration by the soil nails, the soil should not contain a high percentage of cobbles or boulders. Launching nails in ordinary sands, gravel, silts, and clays or mixtures of these is no problem. Penetration will be reduced in dense gravels and stiff clay. A few cobbles and boulders will not be a problem since penetration can still be achieved even if the nail is deflected into another portion of the soil. Nail locations can be adjusted around obstacles to install the correct number of nails. The launcher can easily be repositioned and a replacement nail installed for the nails blocked by subsurface objects.

A "best estimate" of subsurface conditions at the site is necessary to evaluate stability and conduct a preliminary design of nail spacing. The field data form shown in Figure 11(*a*) should be used to note the general soil, rock, vegetation, drainage, grade, and other physical

FIGURE 8 Failing road shoulder typical of those needing stabilization.

factors at the site. An estimate of the subsurface moisture condition at the time when slope movements occurred is essential in the overall evaluation of stability. Engineering geologists or geotechnical engineers should perform the field evaluation and design.

The site factor checklist shown in Figure 11(*b*) contains nail-spacing adjustments for local site conditions. The site factor evaluation is based on local conditions, the confidence in the site condition assessment (probability of sliding), and the consequence of continued slope movement. Generally the "high" site condition deserves a more critical design review (higher site factor of safety, f_s) than the "low" site condition. For high site conditions, it is recommended that a more in-depth site investigation, mathematical slope stability analysis, or both, be performed before a final repair alternative is selected. The site factor checklist is suggested for selecting an appropriate site factor of safety. An example of a completed field data form and site factor checklist is included in the design example discussed in this paper.

Since seepage pressures can have a major effect on the stability of the slope, it is best to install seepage control measures. Drilled horizontal drains and drainage trenches are commonly used to control groundwater and seepage pressures in slopes. Launched horizontal drains can also provide the needed drainage.

A high water table will affect the geometry of the slide, resulting in a larger slide and a lower apparent soil ϕ. Use of launched horizontal drains (Figure 12) and an appropriate apparent ϕ may counter the need to increase the number of nails to account for the

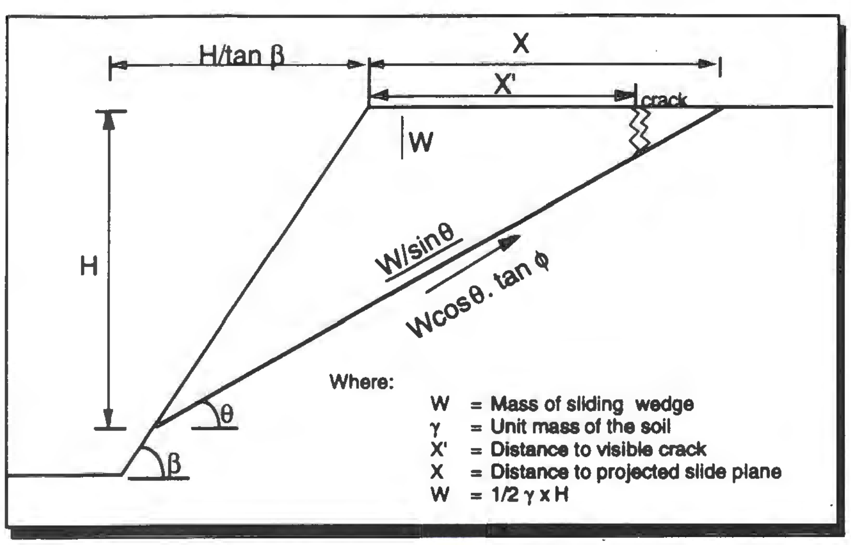


FIGURE 9 Simplified wedge forces.

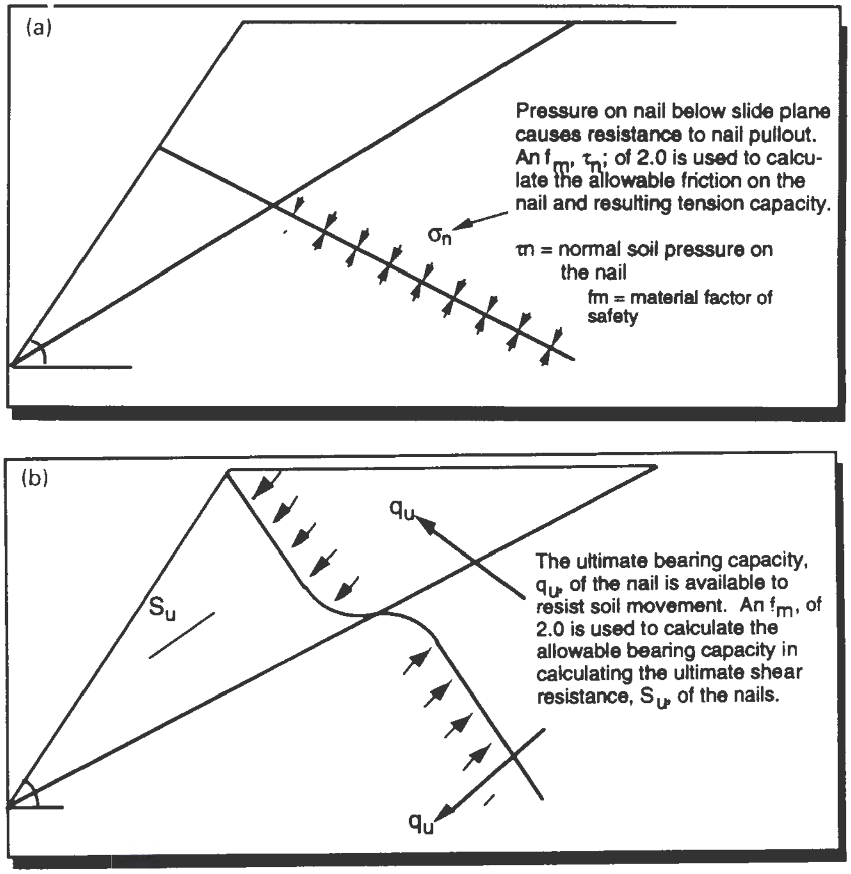


FIGURE 10 Tensile (a) and shear (b) resistance of nail.

groundwater table. This question will be answered as full-scale field installations are completed and monitored. Until then, it is recommended that either the number of nails be increased or the groundwater be controlled in areas with active seepage.

The number of nails, N, From Figure 1 can be adjusted to fit the condition. Although not mathematically exact, adjustments of $0.5N$ for low, $1.0N$ for medium, and $1.5N$ for high conditions will yield overall factors of safety ($f_m + f_n$) of about 1.1, 1.2, and 1.3, respectively. Figure 13(a) shows the preferred diagonal nail pattern.

Design Example

Figures 11(a) and 11(b) show the completed field data forms and site factor checklist for a typical road failure site on an older road in steep, mountainous terrain. The design of the launched soil nail stabilization for this site follows.

Design Information

$$x = 3 \text{ m } (9.9 \text{ ft}) \quad H = 5.5 \text{ m } (18 \text{ ft}) \quad \frac{x}{H} = 0.6 \tag{1}$$

For $\theta = 32$ degrees, use $\phi = 30$ degrees; for $\beta = 42$ degrees, use $\beta = 45$ degrees.

Number of nails per 1 m (3.3 ft) along road shoulder, (from Figure 11): for Curve C, $N = 4$.

Site Factors

The steepness slope factor is contained in the design charts. Groundwater and seepage can be controlled by installing drainage. The other factors are not easily controlled after a road is constructed and must be accounted for during design of the repair. The low, medium, and high site evaluations are judgment calls at best.

(a)

FIELD DATA FORM FOR LAUNCHED SOIL NAILS

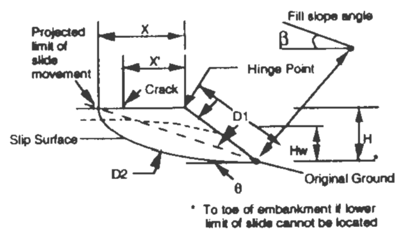
Road Name *Example* Road No. *4671* Date *8/12/92*

Mile Post/Station *MP 3.8* Location T. *7S* R. *7E* Sec. ____

General Site Description: *Steep sidecast fill soil over colluvium (near 90% slope)*

Repair Priority *7* (1-10 High)

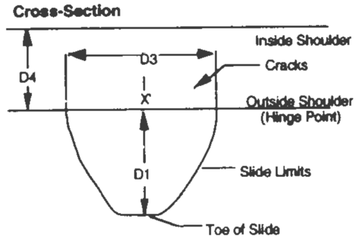
Completed by: *S. Bear*



Dimensions (FT) or (m)

X	*10 ft*	*3m*
X'	*5 ft*	*15 m*
D1	*33 ft*	*10 m*
D2	*5 ft*	*15 m*
D3	*40 ft*	*12.2 m*
D4	*20 ft*	*6.1 m*
H*	*18 ft*	*5.5 m*
Hw	*8 ft*	*2.4 m*
β	*42°*	
ϕ	*32°*	

Cross-Section



Plan

(b)

SITE FACTOR CHECK LIST FOR LAUNCHED SOIL NAILS

SITE FACTORS	EVALUATION						
	Low		Medium		High		Remarks
Steepness of Slope (Slope Ratio)	2:1		1.5:1		1:1	X	
Depth to Failure Surface, D3	<5'	X	5'-10'		10'-15'		
Soil Moisture at Time of Slide	Moist		Wet	X	Seep		
Decayed Logs or Slash Within Fill	None		Some		Many	X	
Soil Type	Sand		Silt	X	Clay		
Consequence of Add'l Failure(s)	Low	X	Med		High		
Potential for Accident or Injury	Low	X	Med		High		

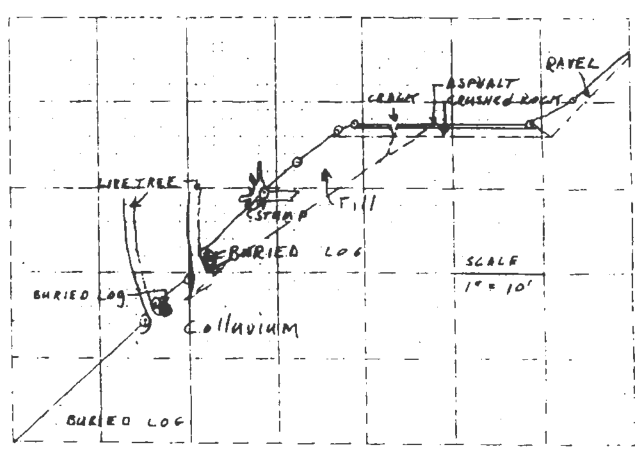


FIGURE 11 Design example field forms: (a) field data form, and (b) site factor checklist.

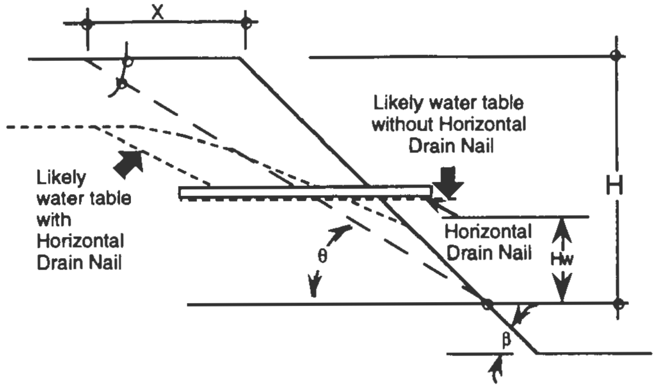


FIGURE 12 Water table consideration.

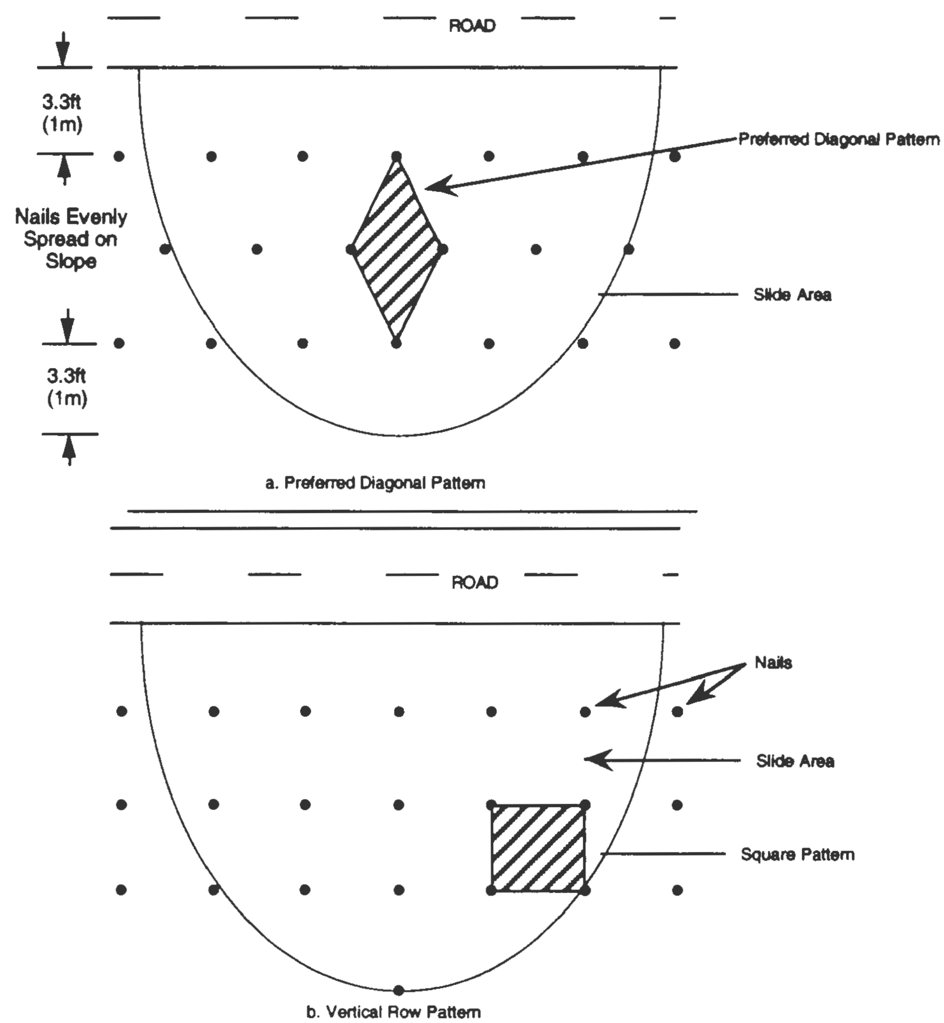


FIGURE 13 Nail pattern.

For this example, the evaluation is tending toward medium:

$$\text{medium} = 1.0N \text{ or } (1.0)\ 4 = 4 \text{ nails per meter (3.3 ft) of road} \qquad (2)$$

Slope Area per Nail

$$\frac{1\ \text{m}\ (D1)}{N} = \frac{1\ \text{m}\ (10\ \text{m})}{4\ \text{nails}} = 2.5\ \text{m}^2/\text{nail}\ (27\ \text{ft}^2/\text{nail}) \qquad (3)$$

Nail Spacing on Slope

$$\sqrt{2.5} = 1.6\ \text{m}\ (5.2\ \text{ft}) \qquad (4)$$

Total Number of Nails

Assuming that the unstable area is within the limits on the sketch and is rectangular with two rows of nails outside the defined site area, the area to be nailed is as follows:

$$(D3 + 3.2)(D1) = (12.2 + 3.2)(10) = 154\ \text{m}^2\ (1{,}656\ \text{ft}^2)\ \frac{\text{Area}}{\text{Area/nail}} = \frac{154}{2.5} = 62 \text{ nails} \qquad (5)$$

Final selection of the number of nails and the nail spacing will depend on the following considerations:

1. The risk and consequence of failure assessed in terms of loss of life, property damage, environmental damage, and traffic disruption (low, medium, or high from Figure 11);
2. The existing stability of the slope and its ability to support the weight of the launcher and excavator (approximately 19 500 kg or 43,000 lb);
3. The sequence of nail installation to enhance the stability of the working area;
4. The maximum depth to the slip surface, perpendicular to the slope surface, not exceeding 4.5 m (15 ft) or 6-m (20-ft) nails;
5. The site factor of safety, f_m, applied relating to the level of confidence in and certainty that the factors will influence the slope's stability;
6. Evaluation of the influence of the groundwater and surface water in the worst-case seasonal condition; and
7. The durability of the nail. Factors that may accelerate corrosion must be appraised. High or low groundwater levels, pH conditions, and the presence of external contaminates such as road salt, organic debris, and leached wastes should be examined. Galvanized steel nails are expected to last as long as galvanized steel culverts under similar conditions.

Cost Estimating and Logistics

The design charts can be used in conjunction with the field data form to estimate the number of nails required. After setup on the site, the launcher is capable of installing 15 nails/hr. A cost range of \$80 to \$135/nail is appropriate for an initial cost estimate for the launched soil nail repair alternative, including mobilization.

Since the excavator normally works from the roadway, minimal site work is usually required for equipment access. On two-lane roads, traffic can usually proceed using traffic control, with full traffic stoppage only during actual launching. Single-lane roads may require longer delays in traffic. The excavator can be moved out of the way for traffic passage after several launches.

The support equipment needed for the soil nail launcher is minimal. The launcher can be moved to a site and set up, launch nails, and move off the site in one day. The launcher can be removed from the excavator's mounting within 30 min. A heavy-duty flatbed trailer or truck is needed to transport the launcher (750 kg or 1,652 lb), rods (54 kg each or 119 lb), and miscellaneous supplies.

Other Potential Applications

Launched soil nailing has many potential applications:

1. Horizontal Drains: Landslides are frequently associated with groundwater and groundwater seeps. Drilled horizontal drains have proven to be effective in reducing or controlling the effect of this groundwater. Launched perforated pipes up to 6 m (20 ft) long have been used to drain local areas.
2. Vertical Gas Vents: Vertical perforated plastic and metal pipes have been used to vent methane gas from landfills. This application has proved fast and safe for the installers.
3. Strengthened Walls: Soil nailing may be used for rapidly adding reinforcement to the materials behind retaining walls to replace deteriorating tiebacks, support increased external loading, support excavation at the toe, and compensate for aging components.
4. Ground Anchors and Tiebacks: With a typical pull-out resistance of 9 to 13.5 kN (2,000 to 3,000 lb), direct pull anchor uses may be limited.

5. Facings and Mesh Holdings: Soil nails may be used to support mesh on rocky slopes and erosion control materials on raveling slopes and fills.

6. Temporary Excavation Support: This method may be used to hold an excavated face until a permanent wall is constructed or while work is completed in the area and backfilled.

7. Widened Roads: The method may also be used to steepen a cut slope or build a small permanent wall at the toe of a cut slope instead of widening the fill, building a retaining structure, or moving into the cut slope.

8. Cut Slope Stabilization: The method may aid in the reinforcement and stabilization of cut slopes.

9. Vertical Drains: Finally, the method can aid in dewatering or consolidating loose materials such as dredge spoils and wet areas under roadways and bridge approaches.

Summary

Launched soil nails have been used to stabilize road- and railroad-related landslides in Europe and the United Kingdom since 1989. The technology was successfully demonstrated in the western United States in 1992.

Design charts for selecting the number of nails required to stabilize small landslides for road shoulders and embankments have been developed using the simplified wedge analysis method. The chart method of design is appropriate for low-volume roads where the cost of geotechnical drilling is generally not warranted. Other methods of design will certainly develop as case history projects are designed and constructed.

The launcher can be used without disturbing established vegetation. With the emphasis now placed on the environment, this method requires little or no vegetation or visual mitigation.

Launched soil nails appear to be an effective, rapid, and practical method for stabilizing small road shoulder landslides. Full-scale design and construction projects are needed to verify the design method proposed in this paper and generate commercial interest in the technology.

References

1. *Application Guide for Launched Soil Nails.* USDA Forest Service, Washington, D.C., July 1994.
2. *Project Report for Launched Soil Nails—1992 Demonstration Project.* USDA Forest Service, Washington, D.C., July 1994.

Stabilizer Mechanisms in Nonstandard Stabilizers

Douglas E. Scholen, *USDA Forest Service, Atlanta, Georgia*

An improved understanding of the mechanism of nonstandard chemical stabilizers is sought in the mineralogy and chemistry of clays and stabilizers. The mineralogy of clays and the chemistry of stabilizers are described. The complete hydrolysis of the rock mineral feldspar produces the rock minerals opaline and gibbsite. This pathway provides a most probable route for the chemical stabilization process. In the natural weathering process, the presence of alkali metal cations prevents completion of hydrolysis and results in an end product of clay minerals except where lateralization occurs. The process of lateralization is paralleled by the effects of chemical stabilizers. Certain organic ring compounds provide a strong attraction for metal cations. Chemical stabilizers contain these compounds in ionized form so that they can remove cations from the clay environment and permit the hydrolysis to proceed to a stage of lateralization. In a certain construction procedure involving injection, development of a high electrochemical potential results in rapid infiltration through normally impervious clays. In another construction procedure limited to scarification, the developed potential is low, and mixing is required. For either procedure, the behavior of resulting stabilized materials resembles that of natural concretions in various stages of lateralization, and the materials are resistant to moisture penetration and frost action. Strength increases with density, which results in higher intermolecular attractions. The physical manifestation of the proposed mechanisms at work has frequently been observed in the case studies, which are referenced in detail for illustration. A review of recent case studies and some supporting laboratory data are provided.

Study F-5, Non-standard Stabilizers, was initiated in 1988 under the FWHA Coordinated Technology Implementation Program (CTIP) to provide information on soil and aggregate stabilizers available on the market but not in general use in the United States. The stabilizers tested are given in Table 1. Previously published case studies show a remarkable project performance for the stabilized aggregates (*1*), and yet laboratory testing indicates only minor changes in engineering properties such as grain size, Atterberg limits, and bearing or shear strength testing. The reason for this apparent contradiction is that chemical stabilization alters the mineralogy of the clay fines to a material with properties that cannot be tested by the standard methods used in highway materials laboratories.

The alteration of clay minerals is but a modification of the weathering processes taking place in nature. Briefly stated, the breakdown of rock minerals by the weathering process, if carried to its ultimate phase, results in the formation of a stable crystalline residue. The presence of alkali metal cations and ionized water in the natural environment stalls this breakdown at an intermediate phase, resulting in formation of clays. In the natural lateralization of clays that results in the formation of pan, iron ore gravels, and bauxite, the cations

TABLE 1 Chemical Stabilizers Tested

NAME	TYPE	USE	200
Condor.SS	Sf nphthln	Subgrade	40%
ISS	Sf nphthln	Subgrade	40%
Road.Bond	Sf limonene	Agg.Surf.	20%
Perma.Zyme	Enzyme	Agg.Surf.	20%
EMC ^2	Bioenzyme	Agg.Surf.	15%
BioCat	Bioenzyme	Agg.Surf	15%

are removed by chelation (2) and leaching, allowing hydrolysis to proceed under certain conditions (*3*). The application of appropriate chemicals to chelate and remove the cations and ionized water will also permit the breakdown to continue to the stable rock and clay mineral phase (2). Appropriate chemicals must be ionized or in enzyme form and contain aromatic rings with a strong negative charge to attract the cations and ionized water (*4*,*2*). These chemicals do not become a part of the rock mineral phase but act solely as catalysts to the breakdown process.

The nature of the structure of the six-carbon benzene ring results in the required negative charge but in the form of an oil (*5*–*7*). Treating this oil with sulfur trioxide and sulfuric acid causes the ring structure to ionize, producing a potential stabilizer (*7*, p. 153). Alternatively, a five- or six-carbon ring may be synthesized into an enzyme system that can accomplish the required result (*5*, p. 755). Thus the carbon ring is the basic building block of the chemical stabilizer.

Occurrence of Clay Minerals

These stabilizers react only with the clay mineral fraction of the soil or aggregate. Understanding the formation and occurrence of clay minerals is essential to understanding the observed performance or lack thereof in the case studies (*1*, p. 11).

Feldspar is the most abundant rock-forming mineral, constitutes 60 percent of the earth's crust, and is the primary source of clay. Clays will not form in arid climates. Clays found in these areas developed during prior climatic periods or were transported. The weathering horizons in a moist climate have ample organic humus to provide the necessary low pH and CO_2 in the soil. The weathering of feldspar results in hydrolysis of the feldspar rock mineral to kaolinite and amorphous silica (*2*,*8*).

During transport in streams and deposition, kaolinite is converted to illite. Kaolinite is an alumina-rich clay, and illite and smectite each have increasing percentages of silica (*2*, p. 25), resulting from changing environment. Illite is rich in potassium (K), whereas smectite is rich in sodium or calcium. Montmorillonite is the predominant type of smectite.

One engineering characteristic that provides an indicator of the mineral content in clays is the activity ratio (AR) (*2*, p. 185; *9*). The AR is the product of the plasticity index (PI) divided by the percent finer than 0.002 mm (2 μm). Anything finer than 2 μm is considered to be clay. Clays with low activity ratios, less than 0.5, are generally kaolinite. Illites have an AR of less than 1.0, and smectites have an AR greater than 1.0. Mixed layer clays will be above or below 1.0 depending on the percentage of each clay mineral present; thus, the history of the deposit is useful in determining whether a clay is illite or mixed layer. Case studies involved a broad range of clays (Table 2).

Clay Chemistry

The basic molecular building blocks of clay (*2*, p. 25) are silica oxide tetrahedra, SiO_4, and alumina hydroxide octahedra, $Al(OH)_6$.

The positive charge of the silica sheet balances the negative charge of the hydrated alumina sheet, and the two sheets form a stable layer except along the periphery of the layer where bonds are broken. This arrangement forms the kaolin clays. Illite and smectite have a silica sheet on either side of the alumina sheet, totaling three sheets per layer. The layers are loosely bonded by the attraction between the H+ of the alumina's hydroxyl and the O− of the silica's oxygen in the adjacent layer (kaolin) or held together by shared cations (illite and smectite). The excess negative charge holds cations and ionized water, H_3O+, to the surface of the layers.

Thus the molecular structure of clays is latticed. The lattice is made up of repeating layers of silica and hydrate of alumina and has charged metal cations, primarily potassium, sodium, and calcium and ionized water attached to the layers. The weaker clays can absorb layers of ionized water among the lattice layers, allowing them to expand and lose density. In an electron micrograph, the clay structure appears like a disorderly pile of club sandwiches in which the layers of bread and meat are the repeating layers of the molecular lattice.

The stability of the lattice is tenuous. The bonding is susceptible to changes in pH; aluminum hydroxide is amphoteric, forming free ions in solution at low and high pH but becoming insoluble and forming the lattice at moderate pH. Silica dioxide forms its lattice when above certain concentrations at low to moderate pH and forms free ions in solution at high pH. The excess negative charge must be balanced by other ions in solution to maintain the stability of the lattice. Because of their differences, the silica can slowly leach out at mod-

TABLE 2 Test Data on Subgrade Soils

LOC.	PRJ	TP	200	2u	1u	LL	PL	PI	AR	CST	ST
...	....	...	...	...	...	...	...	...	...	...	..
MS...	707	UT	82	30	27	50	21	29	1.0	FE	N
..	...	T	..	..	..	43	15	28	..	..	..
TX	204	UT	51	38	36	65	23	42	1.3	INJ	Y
..	...	T	..	..	..	59	20	39	..	..	..
PR	EPL	UT	99	74	73	71	41	30	0.4	INJ	Y
..	...	T	99	76	75	72	36	36	...	...	..
GA	1235	UT	52	26	23	39	23	16	0.6	FE	N
TX	126	UT	42	24	23	65	20	45	1.9	CE	Y
LA	560	UT	96	52	48	66	22	44	0.9	INJ	Y
AL	728	UT	92	56	47	76	31	45	0.8	FE	N
GA	DF	UT	45	16	12	30	23	7	0.4	CE	Y
TN	HW	UT	62	31	23	34	23	11	0.4	CE	Y

Soil sample type TP: treated T, untreated UT.
Equipment used CST: Drawbar farm equipment FE, Construction equipment CE, Injection INJ.
Stable after treatment ST: Yes-Y, No-N

erate pH, but the alumina is held fast by its low solubility.

The key to stabilizing a clay soil lies in removing cations from the lattice and the surrounding environment. The resulting induced cation-free environment causes a spontaneous breakdown into smaller pieces, allowing moisture to drain out and leave the amorphous or crystallized remains of the lattice, which gradually forms a permanently hardened mineral mass.

To accomplish this, a chemical base that has a strong attraction for the positively charged cations and ionized water to be removed, an attraction stronger than that of the clay lattice holding them, is used. Once removed, the cations are permanently disposed of by precipitating as salts or combining with organic molecules so that they cannot return to the solution.

Chemistry of Sulfonated Oil Stabilizers

Sulfonated oils have reacted with sulfuric acid to produce a milder acid, sulfonic acid, one in which the sulfite anion $(SO_2)OH-$ will perform as a base with the hydrogen cation as its conjugate acid. Because some residual sulfuric acid is also present, the stabilizer may be labeled sulfuric acid to comply with shipping regulations.

The oils selected for this purpose are called aromatic oils because their hydrocarbon molecular structure contains one or more rings of six carbon atoms, similar to benzene, that gives them a characteristic aroma. The carbon atom can attach to four other atoms in a molecular structure. The benzene ring has only one hydrogen atom attached to each carbon in addition to the other carbon atoms on either side, leaving an extra electron free at each carbon to form a double bond. This high net (negative) electron density of the ring provides a strong attraction for the (positive) cations on the clay lattice.

Oils do not ionize in solution and therefore cannot react with cations. To produce an ionized solution, the oil must be treated with an oxidizing agent such as sulfuric acid.

The sulfonation process attaches a sulfite ion to the ring, forming a sulfite anion base in conjugate with a hydrogen cation acid (*7*, p. 153; *6*, pp. 508,514). When this solution is diluted to application strength, ion activity is greatly increased and the resulting complex is only mildly acidic. The anions will actively attract cations in their vicinity, moving them away from the clay lattice by chelation (*2,5*). The ring surrounds the cation and holds it by electron sharing between the carbon atoms and the metal atom.

Breakdown of Clay Lattice

Through application of the basic principles of chemistry to the reactants present during the stabilization procedures for a clay soil, chemical reactions between the clays and the stabilizer solution can be inferred and appropriate equations developed to illustrate the process and the observed results. In the case studies, the reactions between the stabilizer solution and the clay fraction were apparent within 48 hr and had achieved major effects within 5 days (*1*, pp. 55,61,62,80).

When applied to the clay, the negatively charged oil/sulfite anions attract the positive cations and ionized water away from the lattice, resulting in a net negative potential toward the outside of the layer and destabilizing the clay lattice consisting of the sandwiched silica and alumina sheets. The substituted aluminum atoms in the silica sheets lose their bonds to the oxygen atoms and are drawn out by the stabilizer anion and hydrolyzed in solution.

The hydrogen cations associated with the stabilizer penetrate the lattice structure to the alumina sheet. As the substituted Al atoms are drawn from the silica lattice, amorphous silica SiO_2 is formed from the silica sheet. The alumina in the alumina sheet is hydrolyzed, forming gibbsite, $Al(OH)_3$. The remaining clay minerals are precipitated as amorphous allophane, which is encrusted with the precipitating gibbsite and amorphous silica.

The injected solution is highly diluted because ionization increases with dilution. Unlike standard stabilizers that actually enter into the final product of the process, these stabilizers act as catalysts to rearrange the molecular structure to a more stable configuration and release bonded moisture. The injected chemical does not become a part of the stabilized clay structure.

Anion Renewal

High ionization is required to promote maximum activity with exchangeable cations. The ratio of exchangeable cations in the clay to active stabilizer anions in the solution may vary from 10^3 to 10^4 depending upon the type of clay. Smectite clays have 10 times the cation exchange capacity (cec) of kaolinite and 5 times the cec of illite (*4*, p. 189). Therefore, the stabilizer anions must act repeatedly, 10,000 times each, and must dispose of the exchangeable cations as soon as they have been removed from the lattice. This is possible because other anions are available in the solution to hold the cations and are not involved in reacting with the clay lattice.

These available anions may be provided by colloidal organic humus present in the soil. Less than 0.4 percent by weight of soil, comparable to an average gray clay shale (*10*), would be required for an average clay content (45 percent). This reaction may be catalyzed by resin catalysts added to the solution. The nature of these resins is proprietary and is not revealed in product literature.

Electrochemistry

Clays have extremely low permeability, and yet the reactions between the stabilizer solutions and the clays were observed to proceed at a rapid rate in all the case studies, indicating an electrokinetic phenomenon at work (*1*, pp. 61,62,65,78,80,85).

In the saturated capillary spaces between parallel flat clay flakes, a concentration of cations develops because of the negative charge on the surfaces of the clay. The negatively charged surfaces and the positively charged cation concentration are called the diffuse double layer. This charged double layer responds to electrokinetic phenomena.

The injection of the stabilizer solution into the clays results in a highly concentrated "ion cloud" at the point of injection. In the ionized solution, positive cations are attracted to the injected negative anions. An electrical potential develops between the injected "ion cloud" and the solution cations. The magnitude of this potential should be a function of conditions developing in the clay layers as a result of the removal of the cations. As the cations are withdrawn from the clay lattice, the lattice develops a net negative charge in the silica sheets, destabilizing the layers. Molecular bonds collapse. The alumina breaks away and is hydrolyzed to gibbsite, and the silica sheet hydrolyzes and decomposes to amorphous silica.

At the time of injection, the voltage gradient is substantial because of the short distance between the ion cloud and unaffected clay and could reach 10 V/cm or more (2 V over 5 mm). This sharp gradient at the ion cloud front results in rapid initial movement of ions into the clay mass surrounding the point of injection.

As the ion cloud moves outward from the point of injection in an expanding cylinder, the concentration of ions is reduced, resulting in a reduction of the voltage gradient and the rate of movement of the ion cloud. Thus, there is a limit to the field of movement. In practical application, this limit has been found to be about a meter from the point of injection for sulfonated naphthalene.

Allowing for a reduction in voltage as the ion cloud travels outward from the point of injection, about a week is required for the ion cloud to traverse a meter of clay. The success of this process has been observed in case studies in Texas, Louisiana, Mississippi (*1*, pp. 59,60), and Puerto Rico (see case study for Carribean National Forest). No failures have been observed or reported.

The limiting distance of ion travel is much less in the scarification method of application in which the solution is applied over a wide area with greatly reduced initial concentration. Instead of full application at a point, as in the injection method, half the same quantity of solution is applied over nearly 4 m^2 in two applications. In this case the voltage gradient developed is small and is lost after a few inches of penetration. Osmotic diffusion plays a greater role in this type of application (*2*, p. 254). Failures were observed in case studies in Mississippi, Alabama (*1*, pp. 13,62), and Georgia when inadequate penetration resulted from using draw bar farm plows instead of hydraulically pressured rippers. Sulfonated naphthalene has been applied by scarification as well as by injection. Sulfonated limonene has been applied only by scarification, as are the enzyme and bioenzyme stabilizers.

Enzymes

Enzymes are the catalysts of biological systems. They not only control the rate of reactions but, by favoring certain geometries in the transition state, can lower the activation energy for the formation of one product from another. The basic structure of enzymes is built of proteins (*5*, p. 753). A typical enzyme consists of a protein chain of over 300 amino acid residues plus a metal cation and has a molecular weight approaching 35,000. The metal cation is located in a cleft of a size and shape that excludes all but certain specific organic molecules whose reactions are to be catalyzed. Various groups and linkages, which include aromatic rings within the enzyme, all work with the metal cation to cleave bonds and form new linkages in intermediate products. These products then react to form the end product and regenerate the original enzyme.

Unlike the inorganic catalysts, enzymes are very specific in the breakdown process. They synthesize certain groups of chemical compounds and limit their action to specific bonds in the compounds with which they react. When the enzymes of a soil stabilizer are mixed with water and applied to the soil, they can act in several ways depending upon their design. These actions may include breaking down the clay lattice and combining cations and other components with organic molecules present. Because of the diversity of action available from enzymes, a combination of several activities from an enzyme stabilizer is possible.

Initial reactions with enzymes are similar to those with sulfonated oils. In some case studies a breakdown of clods was observed in the field during mixing operations (*1*, pp. 61,62,65,78,80,85). The lattice breakdown reduces the size of the clay particles and helps them combine with the organics.

The origin of the stabilizing organic molecules may vary with the type of stabilizer. In the enzyme stabilizer, a higher fraction of silt and clay is required, and the humus present in these fines provides the organic source. In the bioenzyme stabilizer, a bacteria culture that may produce additional organics from the carbon dioxide, nitrogen, and oxygen present in the air is included. Thus, the bioenzyme may be effective with a smaller fraction of clay fines. In either case, the enzyme in the aggregate remains permanently active to deal with additional clays mixed in at a later date (see case study for Lewis and Clark National Forest).

The residual SiO_2 gel and gibbsite serve as a cementing compound that adds strength to the stabilized layer. Other potential cements include insoluble aluminum and iron hydroxides that may develop out of side reactions. Compaction is the key to realizing the effect of these stabilizers. Close contact between soil grains is essential to the cementing process. The greater the compacted density achieved, the more effective the cementing will be.

Mineral Analysis of Samples

A preliminary mineral analysis was arranged through a contract. The analysis included transmission and scanning electron microscopy and x-ray diffraction. Samples of two subgrade soils from the Puerto Rico El Portal site and Texas Road 126, both previously treated with a sulfonated naphthalene stabilizer, were obtained together with samples of the untreated soils taken from adjacent areas. The four samples are currently under analysis, and the results of the treated soils will be compared with the results of the untreated soils in the attempt to detect changes in molecular structure. An extensive program of testing beyond the scope of this study may be required to determine the exact nature of these changes.

Mechanical Effects of Stabilizers

Laboratory testing performed on samples of stabilized aggregates has shown little or no change in grain size distribution and only minor changes in Atterberg limits (Table 2). In normal clays placed in water, the cations diffuse along the exterior but are held close by the negative charge of the clay. Ionized water is attracted and a diffuse double layer of surface charge and adjacent charge develops that attracts and holds moisture (*2*, p. 111). The moisture results in the increase in plasticity noted in clays.

In a number of case studies on treated subgrade soils (*1*, pp. 55,62), the clay fraction hardened irreversibly, leaving the mixture unaffected by moisture and highly resistant to erosion; formerly the mass would have dissolved in a heavy rainfall. This change can result from hydrolysis to gibbsite and opaline.

Gibbsite forms monoclinic (micalike) crystals or spheroidal concretions with a specific gravity of 2.3 to 2.4. Gibbsite is harder than kaolinite and half as hard as quartz. Opaline forms amorphous masses containing 6 to 10 percent water and a specific gravity of 1.8 to 2.3. Opaline is nearly twice as hard as gibbsite and slightly softer than quartz. These encrustations cement the aggregate mass, isolate the clay remnants, and eliminate the attraction of moisture. Both minerals have poor resistance to abrasion and fracture, severely limiting the ultimate strength achievable by this means of stabilization. The major benefit is the increased resistance to moisture assisted by a tensile strength developed from the gel cements.

Rock may be differentiated from soil by its inability to slake (*11*). Pure colloidal rock dust will not develop plasticity with water (2, p. 183). In nature, low-grade metamorphic rocks can contain mixed layers of the micas muscovite and illite (a hydrous mica), which may persist with increasing metamorphic grade (*12,13*). The presence of residual clay mineral does not detract from the rock nature of the material. The clay minerals are bound up and isolated from the effects of any moisture present. When crushed, colloid-sized gibbsite particles (less than 1 μm) with a platy shape resemble and contain similar mixed layer remnants of the chemically active clay minerals. These impart cohesive strength and plasticity when granularized and manipulated with moisture, but to a lesser extent than the pure active clay colloids. The PL of higher AR clays shows some reduction when treated, whereas the PL of a low AR clay shows little change (Table 2). The LL of an active clay is high because the expanding lattice absorbs moisture. The treated colloid has no lattice, reducing the LL. The net PI may remain the same or go either way.

In retrospect it can be seen that altering the clay minerals to silica gel and platy gibbsite that contains remnants of clay structure probably would have little effect on grain size and Atterberg limits in a laboratory sample. The destructive processes of sampling, drying, quartering, and wet and dry sieving all tend to remove any interparticle bonding that has been developed by the gibbsite and silica gel in the compacted layer. Remolding of lateritic soils results in substantial increases in plasticity (2, p. 57). The significant difference between these two rock minerals and the clay minerals is their neutral charge as compared with the negative charge of the clays and their lack of attraction for moisture as compared with the hydrophylic nature of clays. This indifference to moisture, together with the developed tensile strength, is the key to improvements in performance as road surfacing materials.

Project Test Data

Laboratory tests were run on several clay subgrade stabilization projects in which sulfonated naphthalene was used as the stabilizer. (One exception is GA 1235 on which the enzyme stabilizer Permazyme was used; this stabilizer has been successful on many other projects.) Test data are given in Table 2. A broad variety of clay minerals was involved, and initially it was thought that the presence of smectite reduced the effectiveness of the stabilizers. However, contrary to the discussion in the CTIP Final Report (*1*, p. 13), the results of the testing indicated that the only significant variable was the type of construction method used. All projects using draw bar farm equipment (FE-disk plows or chisel plows) or front-mounted grader scarifiers failed to stabilize. Projects using CE-rear mounted, hydraulically operated rippers or scarifiers or injection (INJ) were successfully stabilized. The AR indicates that the clay minerals ranged from kaolinite (AR > 0.5) to illite (0.5 > AR < 1.0) to smectite (AR > 1.0). Uniform, full-depth mixing to a depth of 20 to 30 cm is essential to successful stabilization. In the injection method, subgrade injection on 2-m centers to a depth of 1 m was successful in every case.

Clay Aggregate Mixes

For well-graded aggregates, the minimum PI should be 17 or more or the minimum clay content between 5 and 10 percent. For pitrun aggregates, a potential guideline could require the clay content, determined as the percent finer than 0.002 mm in the hydrometer analysis, to be about one-third to one-tenth of the percentage passing the No. 40 sieve (0.4 mm). The clay flakes form a coating on the grains of silt and fine sand that binds them together permanently with compaction. The thickness of the coating must be adequate to provide strength but not so great as to interfere with a high density. A thickness between 1 and 10 percent of the grain diameter appears to meet these requirements and will result in a clay content of between one-tenth and one-third of the No. 40 material in graded aggregates. The finer quartz silt grains are about 10 μm in dimension, requiring a clay coating 0.1 to 1.0 μm thick. Although this seems thin, 0.1 μm is equivalent to 1000 Angstrom units (A), whereas the stacks of clay platelets may vary in thickness from 30 to 1000 A (2, p. 6).

The lower end of this guideline range, from one-fourth to one-tenth of the No. 40 sieve, should be used with caution in poorly graded mixtures on steeper grades over 6 percent. Poorly graded aggregates fail to develop the uniform surface armor of well-cemented coarse aggregate particles required to resist severe surface erosion. Frequent blading will be required to smooth surface rilling, resulting in loss of stability in the upper aggregate layer.

Effect of Stabilizers on Frost Heave

The soil grain size is critical to the growth of ice lenses (*14*). Clays attract and hold moisture, but their low permeability limits suction. Pure silts have less attraction for holding moisture above the zone of capillary rise but can develop very high capillary suction. A combination of silt with a small percentage of clay should have the greatest potential for frost heave under the conditions of good surface drainage usually encountered in road construction. Enough moisture can be maintained in the surface layers to promote initial formation of ice crystals, and high suction can develop to draw moisture up from a suppressed water table.

Hydrolysis of the clay minerals to hydroneutral, free-draining clay and rock minerals greatly reduces moisture holding and ice lens formation. The treated layer then becomes an overburden layer, which tends to reduce the suction and further reduce the potential for development of ice lenses.

Subgrade stabilization with sulfonated naphthalene was used in Pocatello, Idaho, to stop severe frost heaving (reported over 18 cm) under city streets. There has been no recurrence (*1*, p. 55).

In the mountains of north central Arkansas, on the Ozark National Forest, a number of kilometers were stabilized with pozzolans or enzymes to a depth of 10 cm. This protection was adequate for a mild 10-day freeze common to that area. Freezing has not affected the quality of the stabilized aggregate surfacing (*1*, pp. 71,72).

Laboratory and Field Results

Improving Test Results

Field testing provides the most reliable data. Problems arise in the laboratory. Unless unusual care is exercised, the destructive processes of sampling, drying, quartering, wet and dry sieving, and mixing all tend to remove any interparticle bonding that has been developed by the gibbsite and silica gel in the compacted layer. The significant difference between the alteration products and the original clay minerals is their neutral charge as compared with the negative charge of the latticed clays and their free-draining lack of attraction for moisture compared with the hydrophylic nature of latticed clays. Significant interparticle tensile strength may also be developed. When subjected to established testing procedures, chemically treated clay aggregates are broken apart and have water forced into their interstices in a way that cannot occur naturally because of their acquired cemented hydroneutral nature. Atterberg limits and shrinkage are no longer valid measures of field performance without adjustments in handling procedures. An analogy might be to pulverize a hardened concrete specimen and then compact the remains into a mold to determine the concrete strength.

Agricultural soil tests, such as cation exchange capacity and chemical content and mineral analysis by electron microscopy, may provide useful information on the effect of chemical treatment on clay aggregates by indicating the mineral changes taking place and the potential engineering properties based upon the end materials that develop.

Recent Case Histories

Homochito National Forest, Mississippi

The completion of an embankment stabilization project was discussed in the Final Report (*1*, p. 56). The sidehill embankment was constructed in 1992 on a 3:1 slope in three lifts, injecting the foundation to 4.5 m and each 1.5-m lift with a sulfonated naphthalene solution. In late January 1993, a major subsidence occurred 3 to 6 m downslope from the road shoulder, resulting in a scarp height of 3 m. The scarp slope ratio is approximately 3/4:1. Since then no major changes have occurred. Continued rainfall has had no erosive effect on the scarp or road shoulder. The clay in the scarp has hardened without cracking to firm, smooth material with a mineral consistency and shows no signs of sluffing or other deterioration.

Cherokee National Forest, Tennessee

In September 1991, several areas of subgrade failures on the Hiawassee Camp Ground road were repaired. The pavement was removed, and the clay subgrade was scarified to a 30-cm depth using rear-mounted grader rippers. A sulfonated naphthalene solution was hand sprayed on the subgrade from a 1140-liter tank and mixed in using the grader rippers. The areas were compacted with a steel wheel vibratory roller and chip-sealed. No failures have been observed since treatment.

Chattahoochee National Forest, Georgia

In January 1992, the parking area at the Desoto Falls Camp Ground was completed after one year of delay because of excess subgrade moisture caused by frequent rainfall in the area. The silty clay subgrade was scarified using bulldozer rippers. A water tanker was used to apply the sulfonated naphthalene solution, followed by mixing with the rippers. Portions of the entry ramp were done in three lifts. The treatment induced draining, and compaction was achieved with a steel-wheel roller. The area was then paved with hot mix and has performed satisfactorily since.

Apalachicola National Forest, Florida

In May 1992, 2.7 km of logging access road was surfaced with 10 cm of sandclay, which was rotary mixed with the clay subgrade to a depth of 20 cm. A sulfonated naphthalene solution was sprayed on from a water truck with a front-mounted spray bar and mixed in using a spike harrow facilitated by the rotory mixing. The surface was compacted with a vibratory steel-wheel roller, and an inch of sand was spread to improve traction. The road supports hunter traffic and during July 1993 was used for log hauling. No rutting has occurred through considerable wet weather since construction.

Sabine National Forest, Texas

In September 1992, 1.3 km of Road 126 subgrade was treated with sulfonated naphthalene stabilizer applied by scarification and covered with 8 cm of crushed sandstone aggregate. The road carries heavy oil well servicing traffic and had suffered frequent failures during wet periods. No failures have occurred since treatment.

Carson National Forest, New Mexico

In July 1993, 3 km of the Cerrososo Canyon road was resurfaced. The 10 to 15 cm of existing unbound coarse crushed aggregate was mixed with the silty clay subgrade to a depth of 30 cm using rear-mounted grader rippers. The sulfonated naphthalene solution was sprayed on from a water truck equipped with a side-mounted nozzle. Mixing was performed with the grader rippers. The surface was compacted with a vibratory steel-wheel roller. No raveling has developed on the surface since construction.

Caribbean National Forest, Puerto Rico

In July 1993, a 1-m^2 area proposed for excavation was injected to a 2-m depth with a sulfonated naphthalene solution to determine whether the treated soil could be used for structural backfill. Laboratory testing (Table 2) showed 50 percent clay colloids with a steep compaction curve beyond optimum moisture. Sampled and retested a week following treatment, the treated soil showed improved stability with a flatter compaction curve above optimum but remained highly resilient and was wasted (Table 1).

Ozark National Forest, Arkansas

During September 1992, 19 km of crushed aggregate surfacing was mixed with subgrade silty clay fines and treated with sulfonated D-limonene. Application was by water truck, mixing by blade, and compaction by steel-wheel roller. The roads carry heavy traffic and have required no maintenance since construction. An additional 8 km was completed in October 1993 using similar techniques.

Lewis and Clark National Forest, Montana

Clay shale was added to sandstone aggregate on sections of the Spring Creek Road and stabilized with a bioenzyme stabilizer in 1989 (*1*, p. 84). Corrugations in some areas indicated that the shale percentage was deficient. In 1992, during mixing of additional shale, clods were observed to breakdown rapidly, indicating that the enzyme was still present and active. These sections have been free from washboarding and raveling since, but raveling can be observed on untreated sections. On the Lick Creek and Logging Creek roads, the 1989 bioenzyme treatment of a low PI clayey limestone had little effect for 2 years (*5*, p. 753) but recently has shown less washboarding and potholing than untreated sections.

Ouachita National Forest, Oklahoma

A sandy clay subgrade on the Red Maple road was treated with an enzyme stabilizer in September 1991. Log hauling in 1992 did not rut the surface, although severe rutting occurred on the adjacent collector road.

Oconee National Forest, Georgia

An expansive clay subgrade on Gully Road 1235 was treated with an enzyme stabilizer in October 1991. Depth of treatment was only about 8 cm. Severe rutting occurred during log haul in heavy rains immediately after construction because the subgrade under the treated layer failed.

Kisatchie National Forest, Louisiana

In 1990 and 1991, 6.5 km of severely expansive clay subgrade was injected with Condor SS (*1*, p. 55). Subgrade movement ceased completely, but some local failures in the 40-cm base were attributed to saturation by moisture escaping from the treated subgrade. During an inspection in November 1993, it was discovered that these failures were being caused by fire ant nests under the pavement, which involved continual removal of the oversanded base material because the pavement subsided under traffic.

Conclusions

An extensive literature search on the chemistry of clays and chemical stabilizers indicates that chelation of alkali metal cations occurs following application of certain chemical stabilizer solutions to clays and results in partial lateralization of clay minerals to stable rock minerals. A preliminary mineral analysis of samples of treated clays found that a change in molecular structure did occur following treatment.

REFERENCES

1. Scholen, D. E. *Non-standard Stabilizers*. FHWA-FLP-92-011, Final Report. FHWA, U.S. Department of Transportation, July 1992.
2. Mitchell, J. K. *Fundamentals of Soil Behavior*, 2nd ed. Wiley, New York, 1993.
3. Nahon, D. P. Evolution of Iron Crusts in Tropical Landscapes. In *Rates of Chemical Weathering of Rocks and Minerals*, Academic Press, New York, 1986, p. 173.
4. Grim, R. E. Clay *Minerology*. McGraw Hill, New York, 1968, p. 231.
5. Huheey, J. E. *Inorganic Chemistry*, 2nd ed. Harper & Row, New York, 1978, pp. 542, 457.
6. March, J. *Advanced Organic Chemistry*, 4th ed. Wiley, New York, 1992, p. 36.
7. Smith, M. *Organic Chemistry*, HarperCollins College Outline, New York, 1993, p. 149.
8. Aston, S. R., ed. *Silicon Geochemistry and Biogeochemistry*. Academic Press, New York, 1983, p. 43.
9. Attewell, B. P., and I. R. Farmer. *Principles of Engineering Geology*. Wiley, New York, 1976, p. 176.
10. Potter, P. E., J. B. Maynard, and W. A. Pryor. *Sedimentology of Shale*. Springer-Verlag, New York, 1980, p. 55.
11. Morganstern, N. R., and K. D. Eigenbrod. Classification of Argillaceous Soils and Rocks. *Journal of Geotechnical Engineering Division*, ASCE, Vol 11, No. GT 10, pp. 1137, 1156.
12. Mason, R. *Petrology of the Metamorphic Rocks*. Unwin, Boston, 1978, p. 152.
13. Winkler, H. G. F. *Petrogenesis of Metamorphic Rocks*, 5th ed. Springer-Verlag, New York, 1979, pp. 206, 207.
14. Penner, E. The Mechanism of Frost Heaving in Soils. In *Highway Pavement Design in Frost Areas, A Symposium: Part 1, Basic Considerations*, Highway Research Board Bulletin 225, Washington, D.C., 1959 (University Microfilms International, Ann Arbor, Mich., on demand, pp. 1, 6.)

Bentonite Treatment for Fugitive Dust Control

K.L. Bergeson and S.G. Brocka, *Iowa State University*

A laboratory and field evaluation was conducted of sodium montmorillonite clay (bentonite) as a dust palliative for limestone-surfaced secondary roads. It was postulated that the (negative) electrically charged surfaces of the clay particles could interact with the (positive) charged surfaces of the limestone and act as a bonding agent to agglomerate fine (− No. 200) particulates, and also to bond the fine particulates to larger (+ No. 200) limestone particles. Laboratory testing of limestone fines treated with soda-ash-dispersed bentonite indicated a significant improvement of compressive strength and slaking characteristics. Test roads were constructed in Dallas, Adair, and Tama counties in Iowa using bentonite treatment levels (by weight of aggregate) ranging from 0.5 to 9.0 percent. Quantitative and qualitative periodic evaluations of the roads were conducted with respect to dust generation, crust development, roughness, and braking characteristics. As the bentonite treatment level increased, dust generation decreased. About a 70 percent reduction can be achieved at 9 percent bentonite treatment. Wheelpath crust development is improved. Braking distance and braking handling characteristics under wet surface conditions do not appear to be adversely affected up to the 9 percent treatment level. The bentonite appears to be functioning as a bonding agent to bind small particulates to larger particles and is acting to agglomerate fine particles of limestone. This bonding capability appears recoverable over a wide range of environmental conditions. The bentonite appears to be able to interact with new applications of limestone maintenance material and maintains a dust reduction capability.

The Highway Division of the Iowa Department of Transportation has acted to address "fugitive dust" under research projects HR-151 (*1*) and HR-194 (*2*). This work used a number of different palliatives and proprietary products through laboratory screening and demonstration test sections. A common problem encountered is that many additives are good palliatives but are not cost-effective. The results of past work (*3*) indicate that most fugitive dust that is airborne past right-of-way limits is composed of the fine particulates of silt, clay, and colloidal-sized materials (minus 0.074 mm). Because of the size of these small particulates, the surface area per unit volume is very large. Since all aggregates exhibit a positively or negatively charged surface, the physical chemistry effects occurring between surfaces of fine particulates and chemical dust palliatives are significant. Past work (*1–3*) also indicates that for a dust palliative to be effective, the fine particulates must be flocculated, aggregated, or somehow physically bound to themselves or to larger particulates to prevent them from becoming airborne under traffic.

Results of a recent research project (*4*) indicate that significant dust reduction of fine crushed limestone particulates could be accomplished by a simple treatment with small amounts of bentonite (sodium montmorillonite). The bentonite was mixed with water using sodium carbonate as a dispersing and stabilizing agent and then topically applied. An application rate of 0.4 to 1.0 percent bentonite by weight of dry aggregate resulted in a dust reduction of 70 to 80 percent over un-

TABLE 1 Slaking and Strength Properties, Bentonite-Treated Alden Limestone Fines, Air Cured

	Untreated Control		Bentonite Treated	
Age, days	Slaking Time, minutes	Compressive Strength, kPa[1]	Slaking Time, minutes	Compressive Strength, kPa[1]
4	3	448	255	1240
9	4	345	185	1414
12	2	517	170	1379
14	2	414	180	1207

[1] 1 kPa = 0.145 lb/in^2

treated materials. The mechanism by which the dusting appears to have been reduced is significant. Surfaces of calcium-rich limestone particles are known to be positively charged. It had been postulated that introduction of a material of opposite or negative surface charge might bind the small particulates together. Sodium montmorillonitic clays were selected for use because they possess a negative surface charge. Scanning electron microscope and X-ray chemical dot-mapping of treated materials (*4*) revealed that fine dust particles were preferentially bonded to larger particles, and interparticle bonding was created between larger particles.

The results of this work indicate that bentonite treatment might be an effective dust palliative and stabilizing agent for limestone-surfaced secondary roads.

Laboratory Testing

To evaluate the potential strength of the interparticle physicochemical bonding of bentonite-treated limestone, laboratory testing was conducted to evaluate slaking and strength characteristics. A series of tests was initiated using various curing methods to simulate variations in field exposure of temperature and moisture. Limestone aggregates were obtained from the Alden quarry in northern Iowa. Cube samples 1 in. square were prepared using limestone fines passing the 0.042-mm (No. 40) sieve. The treatment level was a 10 percent solution by weight of bentonite (dispersed with soda ash) applied at a rate of 2 percent bentonite by weight of aggregate. Water was adjusted so the consistency of the mixes was uniform between mixes. Batches of 20 cubes were molded at a time.

Table 1 presents the results (average of at least two specimens) of slaking and strength tests on air-cured samples of untreated and treated limestone fines. Slaking tests were conducted by completely submerging the samples over an open-mesh support and measuring the time it took for the sample to collapse through the mesh. Comparison of the slaking data in Table 1 indicates that the soda-ash-dispersed bentonite samples extended slaking time to 40 to 90 times that of the untreated material. Slaking times typically decreased with age, probably because of loss of moisture from the samples. Compressive strength development using the soda ash as a dispersant was two to four times higher than for untreated material.

To evaluate the influence of temperature on the slaking and strength characteristics, samples were prepared and oven cured at 43.3°C (110°F) for 4 days. Samples were then removed from the oven and allowed to cure additional time both in air and in a desiccator. Table 2 presents the results, which are the average of at least two specimens. Again, fines treated with soda-ash-dispersed bentonite showed significant improvement in slaking and in compressive strength characteristics over the untreated fines. These data implied that bentonite was acting as a stabilizing agent for the fines, and hence might be expected to function as a dust palliative in the field.

Dallas County Test Road

Bentonite is commonly used as a drilling fluid because of its thixotropic properties. It becomes very slippery and sticky when mixed with water. One of the primary concerns on the field projects was the influence of bentonite treatment on braking and handling characteristics and general driving safety.

The test road in Dallas County had a relatively flat topography, few residences, and a traffic count of about 75 vehicles per day (VPD). Average gradation data of loose surfacing material indicated 8 percent passing the 0.074-mm (No. 200) sieve. Test sections were 320 m (1,050 ft) in length and included an untreated control section, 0.5 percent, 1.0 percent, and 1.5 percent bentonite-treated sections. Treatment levels were kept low because of concerns about braking characteristics.

Feed-trade bentonite (sodium montmorillonite) was used for the project. The soda ash (sodium carbonate) dispersing agent was obtained from a regional chemical supplier.

TABLE 2 Slaking and Strength Properties, Bentonite-Treated Limestone Fines, Initial 43.3°C Oven Cure for 4 Days

Added Cure	Untreated Control: Slaking Time, minutes	Untreated Control: Compressive Strength, kPa [1]	Bentonite Treated: Slaking Time, minutes	Bentonite Treated: Compressive Strength, kPa[1]
Air - 1 day	3	69	120	1103
Air - 5 days	3	103	40	1103
Desiccator - 1 day	3	103	120	1276
Desiccator - 5 days	3	103	65	1000

[1] 1 kPa = 0.145 lb/in^2

Two motor graders, operators, and a dump truck was supplied by Dallas County. Two truck-mounted spray distributors were equipped with 880-L (200-gal) per minute centrifugal circulating pumps and had a capacity of 11000 L (2,500 gal) each. The center spray bars were 8 ft long and equipped with eight No. 65 spray nozzles. Six-nozzle, 1.8-m (6-ft) hydraulic spray bars on each side allowed a spraying width selection of 2.4 m (8 ft), 4.3 m (14 ft), or 6.1 m (20 ft). Construction was completed on October 1, 1987.

Construction

For construction of the bentonite sections, all loose surfacing material was tight bladed and windrowed to one side. Several cross-sectional measurements were made of the windrow to estimate the amount of aggregate to be treated. The average of these data indicated approximately 193 t (190 tons) per mile of loose surfacing materials.

Bentonite solutions were field-mixed in the distributors in 5500-L (1,250-gal) batches at a 7.5 percent bentonite solution (by weight of water) concentration. The batch formula used 5500 L of water, 22.7 kg (50 lb) of soda ash, and 340 kg (750 lb) of bentonite. Field mixing was accomplished by connecting a 3-in.-diameter hose to the back of the distributor, which was then discharged into the top access port. The soda ash was slowly added by hand-pouring directly into the discharge stream, and was allowed to circulate approximately 10 min. The bags of bentonite (22.7 kg) were then slowly added to the discharge stream until 340 kg (750 lb) had been incorporated. The bentonite was allowed to circulate and mix for an additional 30 min. In general, field mixing and application of the bentonite solution proceeded very well with the conventional equipment.

For application the windrow was spread out to a width of approximately 2.4 m (8 ft) on half of the road. The distributor, using the center 2.4-m (8-ft) spray bar, applied about one-fourth of the solution in the first pass. Immediately behind the distributor, one patrol bladed the treated aggregate to a windrow in the center of the road. The following patrol spread the windrow to a width of 2.4 m (8 ft) on the opposite side of the road. The distributor then applied another one-fourth of the solution, and the process continued until the required amount of solution had been incorporated with the surfacing. Final blade mixing was accomplished with two passes of both patrols. One final pass was made to spread the material over the surface for traffic compaction. After construction, normal routine maintenance blading practice was followed for the duration of the project.

The medium bentonite-treated section (1.0 percent by dry weight of aggregate) required 11000 L (2,500 gal) of the 7.5 percent bentonite solution. The treated material was damp to wet. Treatment levels above 1.5 percent bentonite, using spray distribution methods, would have required drying between applications.

Field Testing

Field testing was conducted from October 1987 through August 1989 and consisted of fugitive dust tests (in and out of wheelpaths) using high-volume air sampling of dust generation under traffic and braking tests to evaluate the influence of treatment on stopping distances and handling characteristics.

Two high-volume stationary air samplers manufactured by General Metal Works were used for collecting dust. The high-volume sampler is based on gravimetric principles and capable of sampling large volumes of air for the collection of suspended particulate matter as small as 0.01 mm in diameter (*5*). Glass fiber filters were used for collection. For testing, both air samplers were placed in the center of each test section, one on each side of the road. The sampler blower motors were powered by a gas generator. Ten passes of a vehicle traveling at 64 to 72 kph (40 to 45 mph) were made between the samplers for each test. The filters with the collected dust were sealed in the field and returned to the laboratory for testing.

Testing was conducted periodically over a wide range of maintenance grading conditions, from well-developed wheelpaths to immediately after grading. Test results represent actual service conditions along with their inherent variability.

Figure 1 presents the results of wheelpath dust data over the project duration for the section with 1.5 percent bentonite. Dust generation is expressed as positive (increase in dust) or negative (decrease in dust) relative to the untreated control section. The control section is represented by the horizontal line at zero dust generation in Figure 1. The overall average of these data indicates about a 19 percent long-term (>2 seasons) reduction in dust generation. Dallas County applied 305 t (300 tons) of new limestone maintenance surfacing over the test road in August 1988, approximately 290 days after construction. This material was end-dumped and spread over the surface without blending. Test data indicate that the bentonite interacted with this new surfacing material and continued to function to reduce dust generation. Results shown in Figure 1 are typical for the treated sections. The 0.5 and 1.0 percent treatment levels reduced dust on the order of 5 percent over the project period. The total data set is presented in the final report for HR-297 (6).

The braking tests were accomplished using a half-ton pickup. The test was conducted by locking the brakes at a speed of 40 kph (25 mph). The braking distance was measured from the start of the skid mark to the center of the front wheels of the truck. Figure 2 shows the wheelpath braking test data for the Dallas County road. Ten tests were conducted under dry conditions; however, only four tests could be conducted under wet conditions. The average variation in results for the dry surface was considerably greater than that for the wet surface. This is believed to be due principally to the wide range of surfacing maintenance conditions under which tests were conducted as well as to the limited data set for the wet-surface tests. Although there are not enough data to be statistically significant, there did not appear to be any major differences in braking distance between the various test sections. The out-of-the-wheelpath tests show trends similar to the wheelpath tests for all bentonite treatment sections. These results indicated no apparent adverse effect on braking characteristics for the various treatments as compared with the untreated section.

Scanning Electron Microscopy

In order to determine if particle-to-particle bonding was taking place with bentonite treatment, samples of the

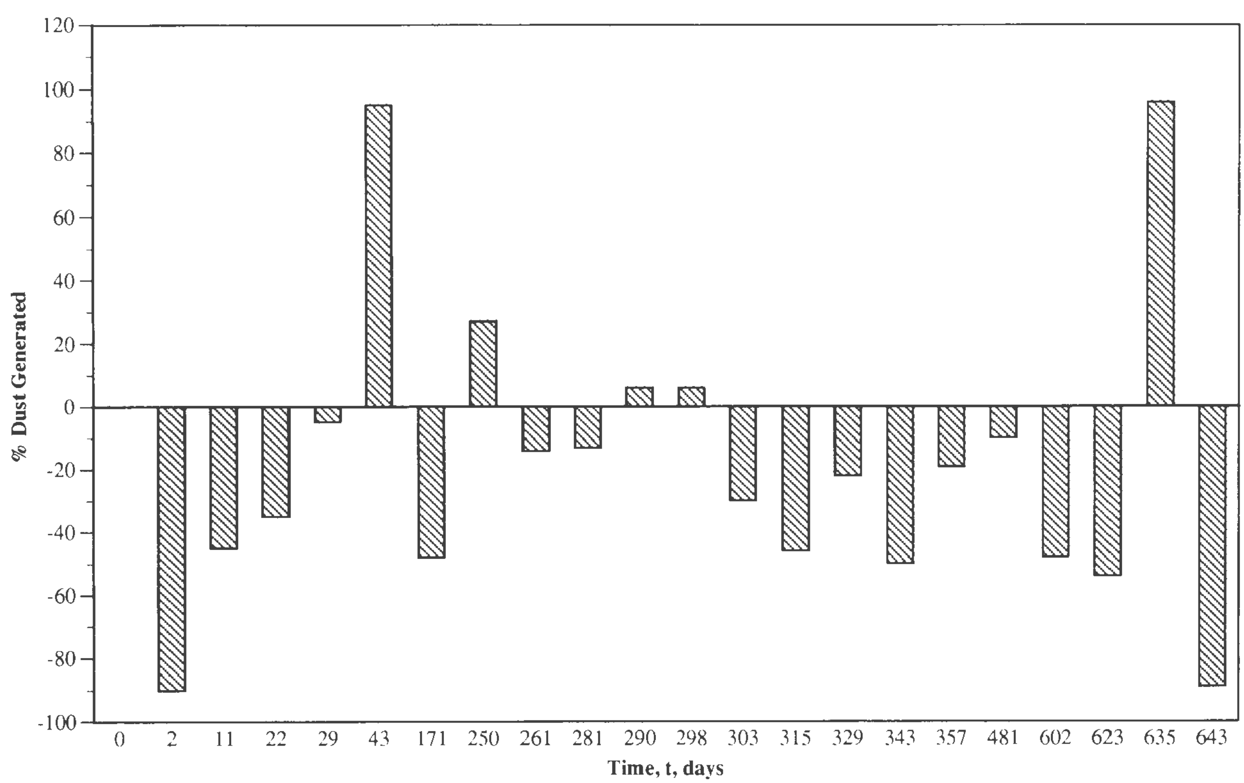


FIGURE 1 Dallas County test road dust generation for 1.5 percent bentonite treatment in wheelpaths.

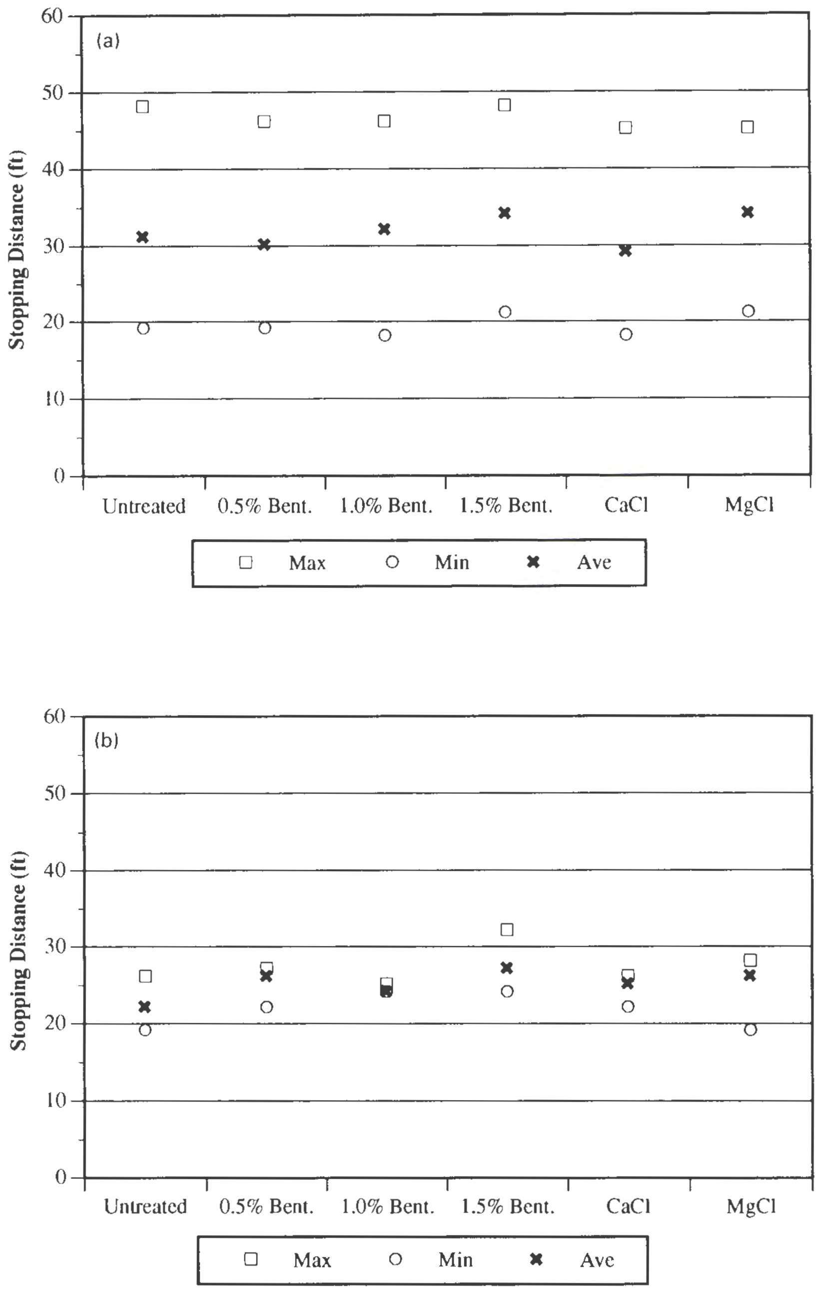
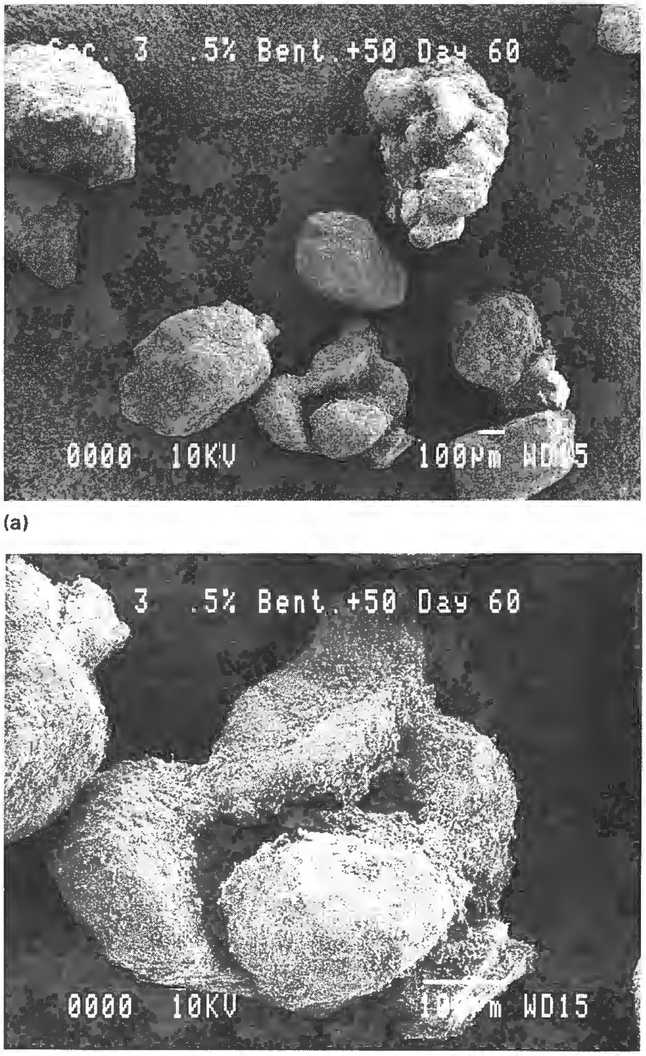


FIGURE 2 Wheelpath braking distance results for Dallas County test road on a dry surface (*a*) and on a wet surface (*b*).

material retained on the No. 50 sieve were mounted for scanning electron microscopy (SEM) analysis. Typical results are shown in Figure 3. Figure 3(*b*) is a threefold magnification of the agglomerated particle shown at the bottom center in (*a*). Scale bars are shown in the lower right corners. Particle-to-particle bonding and agglomeration of fines is evident. These results are typical of numerous SEM observations and study of samples from the No. 50, No. 200, and the minus No. 200 sieve fractions obtained at various periods throughout the project.

Adair County Test Road

Although the dust reductions observed in Dallas County were not dramatic, results were encouraging enough from the longevity of reduction and braking character-



(a)

(b)

FIGURE 3 SEM photographs of material retained on No. 50 sieve from 0.5 percent bentonite-treated section at 60 days after construction.

istics that an additional test road was constructed using up to 3.0 percent bentonite treatment in Adair County.

Based on the experience in Dallas County, the construction method was altered to incorporate higher percentages of bentonite treatment levels; therefore, a dry bentonite application method was developed. The Adair County test road traffic is about 80 VPD. Construction was completed in the first part of July 1989.

Construction

Before construction, the test road had been prepared by tight-blading the loose road surface material to a windrow on one side. Several cross-sectional measurements were taken and averaged for each section in order to determine the amount of bentonite needed for each treatment. The 22.7-kg (50-lb) bags of bentonite were then distributed along the measured and marked windrow in each test section and hand applied. The bentonite and the loose limestone surfacing were thoroughly dry-mixed by four passes of the two graders and finally spread in an 8-ft-wide layer on one side of the road.

The soda ash was added to the tank truck water at the rate of 9.1 kg (20 lb) per 2200 L (500 gal) of water and thoroughly mixed by the circulating pump. The water with soda ash was spray-applied to the 8-ft-wide strip of bentonite-treated limestone and simultaneously road-mixed by two motor graders working in tandem. Application of water with soda ash continued until a consistency of about a 7.6-cm (3-in.) slump concrete was achieved. After final mixing the material was spread uniformly over the road surface for traffic compaction. Normal periodic maintenance blading was resumed after construction.

Field Testing

Testing was conducted from July through November 1989. Dust data collected from this test road showed a significant increase in dust reduction at the higher percentages of bentonite treatment. Figure 4 shows the wheelpath dust data for the 3.0 percent bentonite-treated sections as compared with the untreated control section. An average dust reduction of about 41 percent was observed over the test period. The 1.5 percent treatment exhibited an average reduction of about 17 percent compared with the 19 percent reduction observed in Dallas County. The complete test data for the Adair test road are given in the final report on the project (*6*).

Results of the braking tests are shown in Figure 5 for the wheelpath testing. Fourteen tests were conducted on dry surfaces, and six tests were conducted on a wet surface. As with the Dallas County test road, results on the dry surface were more variable than those on the wet surface; however, no discernible trend appeared evident that would indicate an adverse effect on braking distance of bentonite treatment up to 3.0 percent.

Scanning Electron Microscopy

Several samples of material retained and passing the various fine sieve fractions were investigated using the SEM. Aggregation of particulates was routinely evident in bentonite-treated samples and not evident in un-

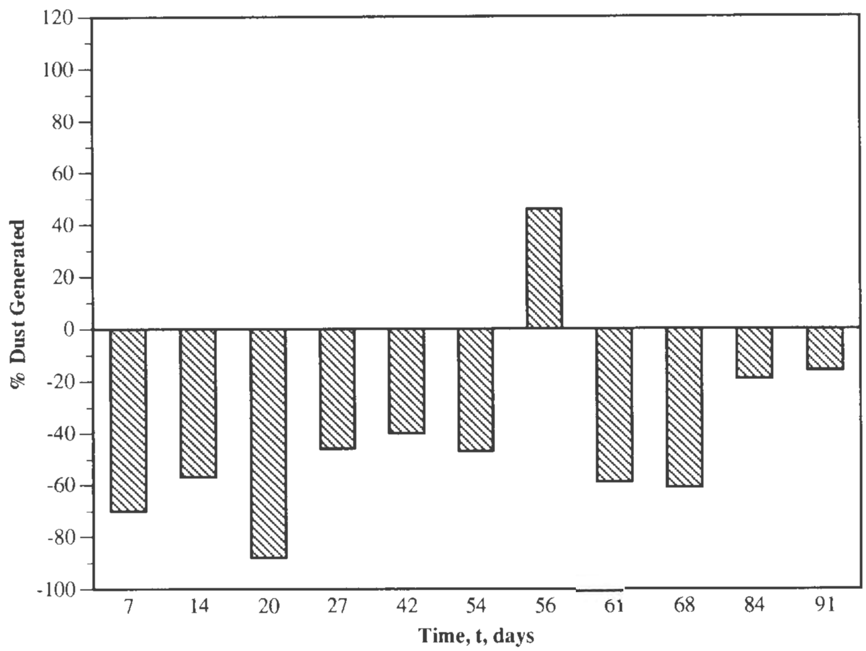


FIGURE 4 Adair County test road dust generation for 3.0 percent bentonite treatment.

treated samples, as was seen in the Dallas County photographs.

Tama County Test Road

Based on the results and experience of the Dallas and Adair county test roads, a project was initiated in Tama County that incorporated 3, 5, 7, and 9 percent bentonite treatment levels. Construction was by the dry application method used in Adair County, and proceeded rapidly. It was completed in 1 day under traffic.

Balling up or agglomeration of the bentonite was observed beginning with the 5 percent bentonite treatment section during the wet mixing process. This balling effect became more pronounced (but not drastic) as the treatment level increased, and was most apparent in the 9 percent section. Based on field observations and review of the lab data, the following changes in procedure are recommended for future construction: (*a*) use a soda ash solution concentrated (by weight of water) at one-tenth of the bentonite treatment percentage, and (*b*) alter the construction wet mixing procedure by saturating the dry bentonite-mixed surfacing material with soda ash solution before the first wet mixing pass of the patrol.

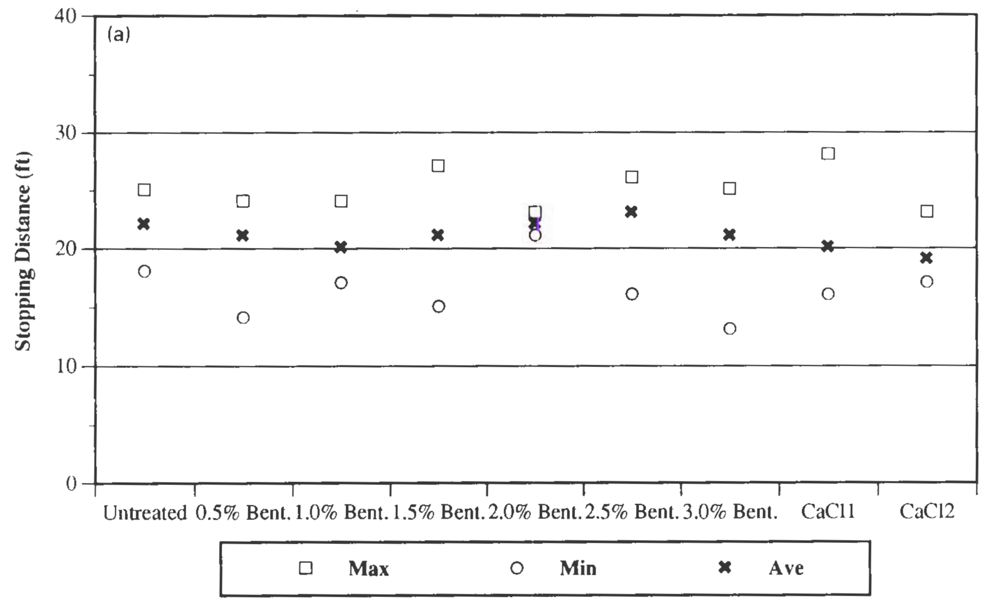
The fact that the sections treated with 7 and 9 percent did not get the bentonite as well dispersed throughout the material may have reduced its effectiveness.

Field Evaluation

With bentonite treatment, dust is reduced but is still being generated after treatment. Bentonite treatment does not achieve as dramatic results as chloride treatments, and the traveling public may not perceive a reduction. Therefore field testing on the Tama County test road consisted of qualitative evaluations by a panel.

Field testing consisted of qualitative evaluation of dust generation, crust development, and roughness and braking characteristics. Evaluations were conducted periodically and independently by a panel composed of personnel representing Marshall County, Tama County, the Iowa Department of Transportation, and Iowa State University (ISU).

An evaluation form developed by ISU was used by all the panelists for their observations. The form contained the following information: weather conditions (day of and day before evaluation); maintenance conditions; and surfacing material conditions, dry, damp, or wet. The panel evaluated the amount of dust generation for each bentonite-treated section compared with the control section and evaluated the crust development and roughness of each section. The dust generation was expressed as a percentage of the control, with the control having a value of 100 percent. The crust development and roughness were evaluated on a rating system from 0 to 5 with 0 being the worst and



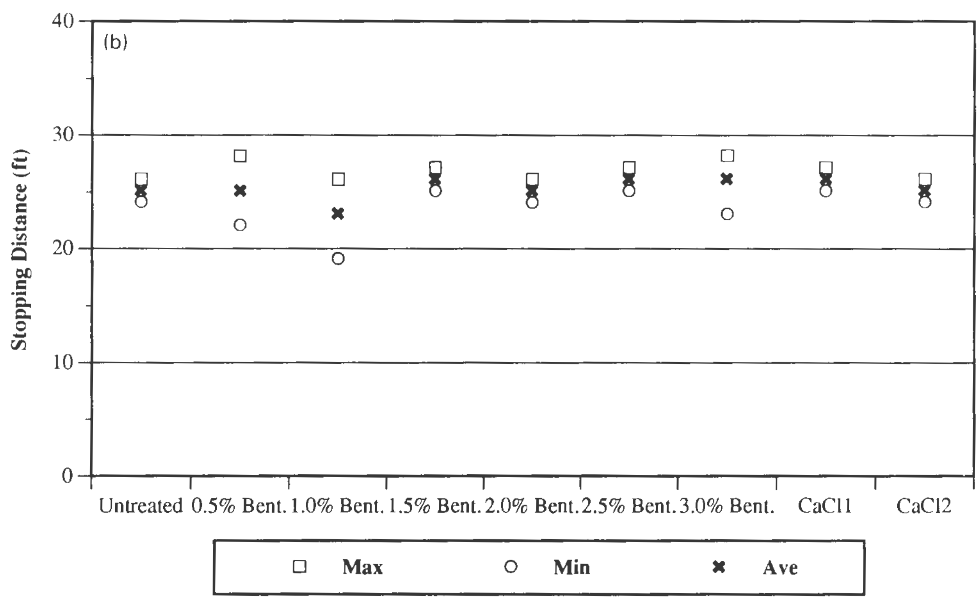


FIGURE 5 Wheelpath braking distance results for Adair County test road on a dry surface (*a*) and on a wet surface (*b*).

5 being the best rating. Results are shown in Table 3. Figures 6 and 7 show the data in graphical form.

The dust generation data indicated a relatively uniform standard deviation of plus or minus 11 percent at a 67 percent level of significance for all test sections. The data shown in Figure 6 must be viewed with this in mind. From Figure 6, the 3 percent bentonite treatment results in a dust reduction of about 45 percent, which compares roughly with the 41 percent quantitative wheelpath reduction observed for the same treatment level at the Adair County test road under the HR-297 project (6).

At a 9 percent treatment level, dust reduction is on the order of 70 percent. These qualitative data indicate that bentonite treatment is effective in reducing dust generation. It should be noted that the data for the 7 and 9 percent treatment may not reflect actual dust reduction capability because of inadequate dispersion of

TABLE 3 Evaluation Averages and Standard Deviations for Dry Surfacing Material Observations

Evaluation Category	Percent Bentonite	Average (n=19)	Std.Deviation (n=19)
Dust Generation	0	100	0
	3	55.0	10.9
	5	44.3	11.1
	7	39.1	10.4
	9	32.1	11.0
Crust Development	0	1.37	1.27
	3	2.21	1.36
	5	3.16	1.50
	7	3.11	1.45
	9	3.05	1.73
Roughness	0	3.74	1.33
	3	3.95	0.69
	5	3.95	0.83
	7	3.68	0.73
	9	3.21	1.10

the bentonite during the construction process. With a standard deviation of ±11 percent, differences between treated sections may be subject to question.

Crust development observation data given in Table 3 are shown graphically in Figure 7. These data indicate that the 3 percent treated section had somewhat better crust development than the control. The 5, 7, and 9 percent sections appeared to be relatively even, with values about two times better than the control. Good crust development would be expected to extend the life of the surfacing material. Again the standard deviation is about ±1.5, and differences may be questionable.

There was no clear trend evident with respect to roughness from Figure 7. All sections exhibited a rating from 3 to 4.

A major concern at the start of this project was the influence of high levels of bentonite treatment on braking characteristics and safety. Tama County and ISU personnel were at the site and conducted braking tests under wet to very wet conditions independently and at different times. There were no discernible differences in braking distance or handling characteristics during braking on any of the sections.

Scanning Electron Microscopy

Samples from the minus No. 200 sieve fraction from the control section and the 3 and 9 percent bentonite treatment sections were prepared for SEM micrographing. The bonding of the fines to the larger particulates was evident, as was the agglomeration of the fines.

Current Research

Field evaluation of the Tama County test road is ongoing. During the summer of 1993, a test road was constructed in Appanoose County incorporating bentonite treatment levels of up to 12 percent. Field evaluation there is also ongoing. Preliminary data indicate that braking and handling characteristics are not dastically affected at the 12 percent treatment level.

Economic Considerations

Table 4 summarizes construction cost breakdown data for materials and installation of bentonite treatment. Equipment and labor costs were the 1993 costs provided by the counties and assume construction of 1.6 km (1 mi) per day, which is conservative. Materials costs are FOB costs to the Des Moines, Iowa, area.

Cost of a 38 percent concentration calcium chloride treatment at an application rate of 0.95 L (0.25 gal) per square yard was approximately $1,600 per 1.6 km (1 mi) in 1993. In Iowa, a minimum of two applications per summer is normally required for dust control. This yields a minimum cost of about $3,200 per 1.6 km per year. The 9 percent bentonite treatment cost is about $1,750 per 1.6 km per year, assuming that the bentonite is effective for only one season. This is believed to be a conservative comparison since the longevity of dust reduction using bentonite has not yet been firmly established. Current data indicate that a single treatment acts to reduce dust over an extended time period (>2 seasons).

Conclusions

The results of this research indicate that bentonite treatment of limestone-surfaced secondary roads is a cost-effective dust reduction treatment. Data from the Dallas, Adair, and Tama county test roads indicate the following:

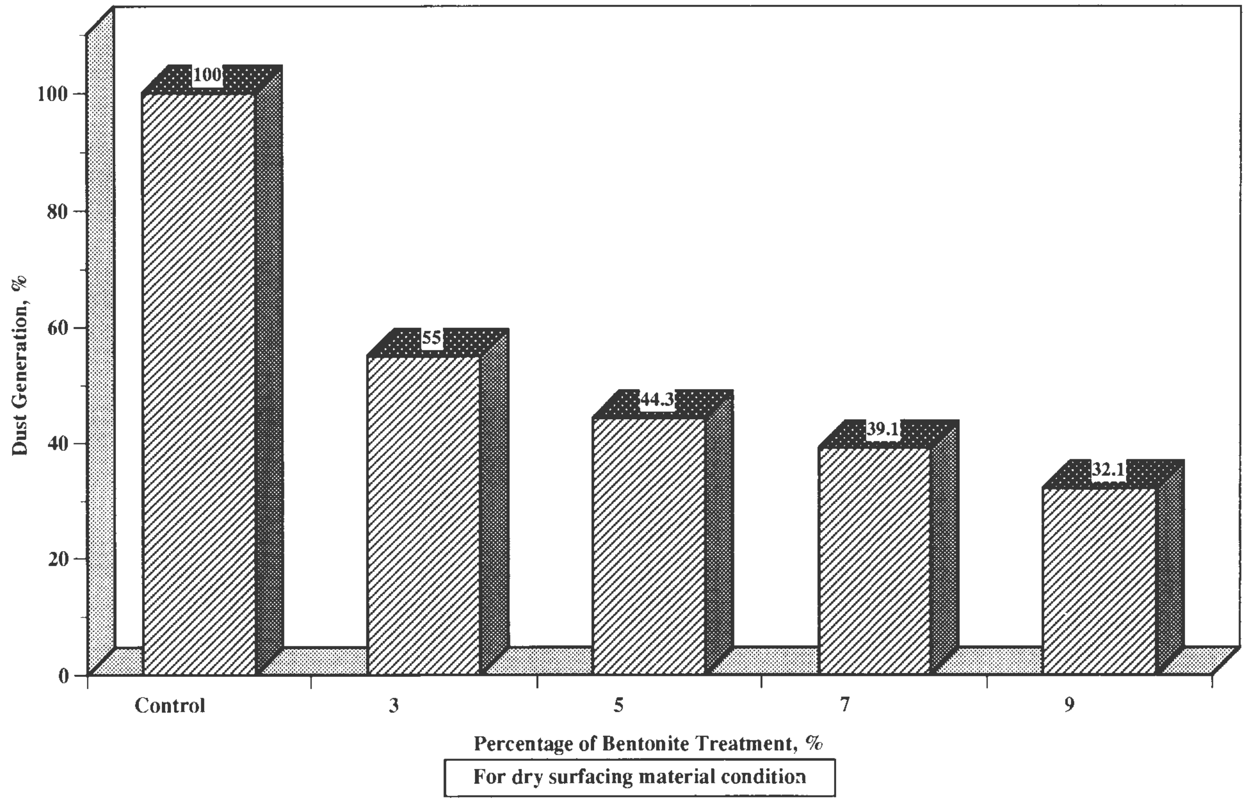


FIGURE 6 Average amount of dust generation for each level of bentonite as compared with control section.

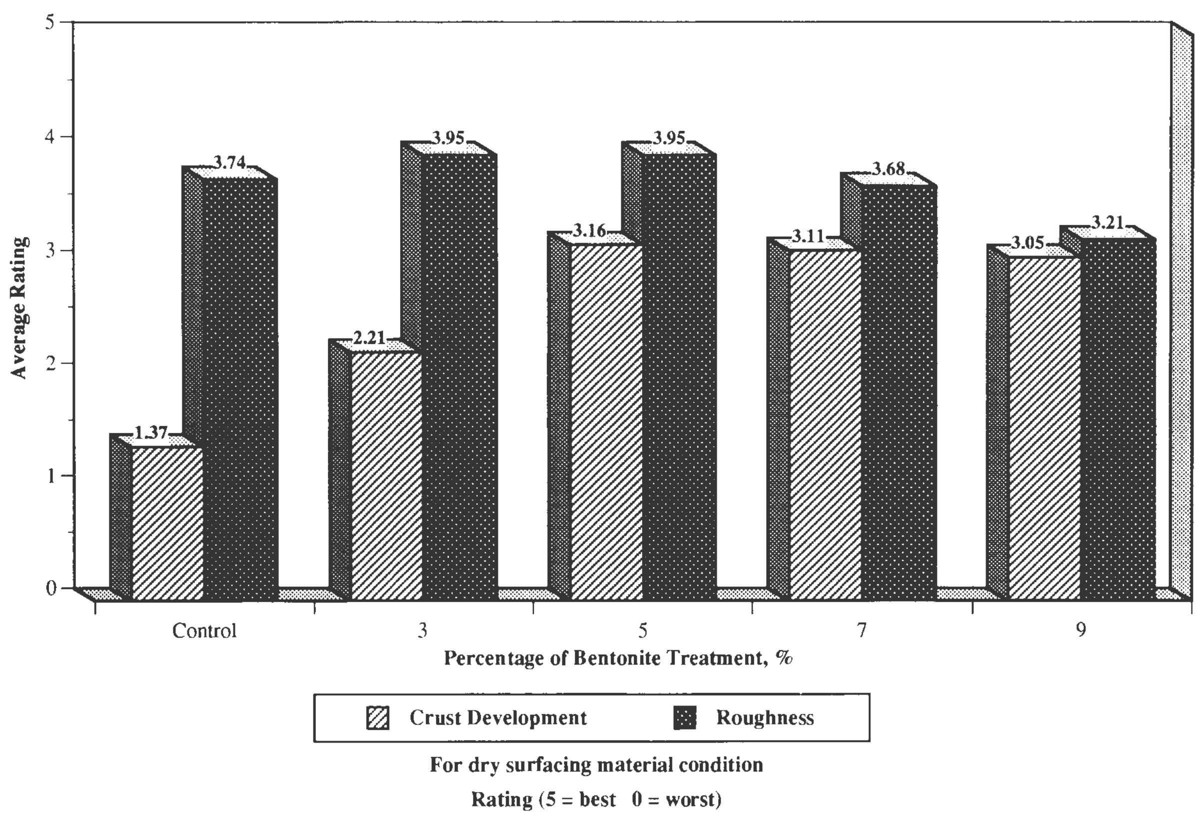


FIGURE 7 Average crust development and roughness ratings for each level of bentonite treatment.

TABLE 4 Breakdown of Construction Costs per 1.6 km, 1993 Cost Data

	3% Bentonite	5% Bentonite	7% Bentonite	9% Bentonite
	Cost in dollars			
Bentonite	300	450	795	1,375
Soda Ash	30	35	50	60
Water	35	40	50	70
Grader + operator	340	340	340	340
Tandem truck + operator	40	40	40	40
Water truck + operator	200	200	200	200
Actual Cost of Construction[a]	945	1,105	1,475	2,085
Normalized Cost [b]	990	1,230	1,480	1,740

[a]Estimated Amount of Limestone Present: 3% section ⇒ 111 t/mile (109 ton/mile)
5% section ⇒ 100 t/mile (98 ton/mile)
7% section ⇒ 126 t/mile (124 ton/mile)
9% section ⇒ 170 t/mile (167 ton/mile)

[b]Normalized Costs Based on a Typical Secondary Road Average of 127 t/mile (125 ton/mile) of Loose Limestone Surfacing Material.

- Bentonite treatment levels from 5 to 9 percent by weight of aggregate may provide cost-effective dust reductions ranging from about 50 to 70 percent, respectively;
- Dust reduction is believed to be accomplished primarily by the function of bentonite as a surface-active bonding agent to bind small particles to larger particles and agglomerate the fine (minus No. 200) particulates;
- The dust reduction mechanism appears recoverable from a wide range of environmental service conditions and remains effective for more than one season;
- The bentonite appears able to interact with limestone maintenance surfacing applied after treatment to maintain a dust reduction capability; and
- Braking and handling characteristics do not appear to be adversely affected up to the 9 percent treatment level.

Benefits

The use of bentonite as a dust reduction treatment appears to have the following benefits:

- It is low cost, readily available, naturally occurring, and environmentally sound;
- It is suitable for rapid construction under traffic using conventional equipment and county personnel;
- Normal county maintenance blading practices can be followed; and
- It displays long-term effectiveness.

A disadvantage of bentonite treatment is that it does not result in the same initial dramatic dust reduction as chloride treatment. Although dust is significantly reduced, it is still generated, and the traveling public may not perceive a reduction.

ACKNOWLEDGMENTS

This research was sponsored by the Highway Division of the Iowa Department of Transportation and the Iowa Highway Research Advisory Board under Iowa DOT Projects HR-297 and HR-351.

The cooperation and assistance of the Adair, Dallas, and Tama county engineers and their personnel are gratefully acknowledged. Special thanks to Gary Harris, research engineer for the Iowa DOT, for his assistance.

REFERENCES

1. Hoover, J. M., K. L. Bergeson, D. E. Fox, C. K. Denny, and R. L. Handy. *Surface Improvement and Dust Palliation of Unpaved Secondary Roads and Streets.* Final Report, Research Project HR-151. Iowa Department of Transportation, Ames, July 1973.
2. Hoover, J. M., J. M. Pitt, M. T. Lustig, and D. E. Fox. *Mission Oriented Dust Control and Surface Improvements Processes for Unpaved Roads.* Final Report, Research Project HR-194. Iowa Department of Transportation, Ames, May 1981.
3. Handy, R. L., J. M. Hoover, K. L. Bergeson, and D. E. Fox. Unpaved Roads as Sources for Fugitive Dust. *TR News Vol. 60*, 1975, pp. 6–9.
4. Oren, G., T. Demirel, and D. Boylan. *Reducing Dusting of Limestone Fines.* Final Report. Iowa Limestone Company, Des Moines, January 1985.
5. H. E. Hesketh and M. S. El-Shobokshy. *Predicting and Measuring Fugitive Dust.* Technomic Publishing Company, Inc., 1985.
6. Bergeson, K. L., and A. M. Wahbeh. *Development of an Economic Dust Palliative for Limestone Surfaced Secondary Roads*, Final Report, Research Project HR-297. Iowa Department of Transportation, Ames, Feb. 1990.

The opinions, findings, and conclusions expressed in this paper are those of the authors and not necessarily those of the Highway Division of the Iowa Department of Transportation.

SURFACING

Bituminous Pavement Design and Construction for Low-Volume Roads in Cold Climates

Alan Forsberg, *Blue Earth County, Minnesota, Highway Department*

Blue Earth County in Minnesota has developed a process for bituminous pavement design and construction for low-volume roads in cold climates. Principal features of the process include cost-effectiveness, quality pavements, and the flexibility to adapt the design to the particular road segment. The process allows separate payment for asphalt cement and aggregate. However, the contractor must pay for any asphalt cement over a given percentage. This requires the contractor to balance the quality and gradation of aggregate against the cost of providing asphalt over the given maximum amount. Construction inspection remains the county's responsibility. This typically saves $0.50 per ton added to contracts for the quality management approach where design and testing responsibilities lie with the contractor. Key features necessary to ensure quality pavements include approval of a trial mix by Minnesota Department of Transportation, provision of a nuclear density gauge and roll test patterns to determine the maximum density compaction process, regular gradation and asphalt cement content tests to ensure that a proper blend of quality materials is used, regular voids content tests during construction, and trimming of the aggregate base by use of automatic controls on a motor grader to ensure good ride quality and a uniform aggregate base and pavement section. The process described has evolved over the last 5 years. There is no quantitative measure of results. Qualitatively, results appear to be good. In addition, the concepts appear to be consistent with the Super Pave Level I concepts of the Strategic Highway Research Program now being implemented.

The emphasis in this paper is on bituminous pavements for low-volume roads in cold climates typical of rural county highway departments in the northern United States. An integrated design and construction method for bituminous pavements is discussed that controls the air voids content appropriate for the particular road by varying the gradation of aggregates and asphalt content.

Construction of quality bituminous pavements requires a systems approach. In concept, this is simple. The product—consisting of appropriate aggregates mixed with a relatively small amount of asphalt, placed on a prepared base, and compacted—provides a serviceable pavement. However, due to the large number of variables and demanding environment, execution can be very complex and filled with risks.

Because the system relationships of a quality pavement are critical to success, a brief discussion of the planning, design, construction, and maintenance of these pavements will provide a proper context.

It was also noted during research for this paper that the basic concepts of designing and constructing bituminous pavements have been under consideration for several decades. It is hoped that the Strategic Highway

Research Program (SHRP) will synthesize the work done to date and advance the process toward science.

Background

The paper describes an empirically based method that has worked for Blue Earth County, Minnesota. Blue Earth County has 720 mi of highway divided about equally among local, minor collector, and major collector by functional classification. Average daily traffic ranges from under 100 to several thousand vehicles. About 400 mi is paved; the rest is aggregate surfaced. The population is slowly declining in the rural areas of the county; traffic volumes in these areas are relatively stable. Population and traffic volumes around Mankato, a growing regional center, are both steadily increasing.

The method described here may be inappropriate for larger, high-traffic-volume agencies. Also, small agencies typically cannot economically control some critical variables, such as asphalt cement grade and quality, available aggregates, and state and federal aid regulations. The typical county engineer needs the wisdom to accept those variables that cannot be changed and to optimize those that can. The local road agency engineer must be pragmatic in view of his or her immediate accountability to the governing body and the public.

Systems Approach to Bituminous Pavement

Quality bituminous pavements can be achieved only by pursuing a systems approach, including planning, design, construction, and operation and maintenance. Planning includes the selection of the roads to pave considering their present and future role in the road system. Design includes the selection of the type of pavement and required structural strength. Construction includes the contracting, inspection, and administrative process. Finally, effective operation and maintenance are essential to maximize the return on the investment in the pavement.

Large agencies frequently have the resources to research and develop each of the stages in detail. However, they also risk losing the relationship between these stages. Smaller agencies are often forced to operate as a system since one engineer administers and directs the entire program. However, Blue Earth County lacks the expertise to understand each stage in detail and must rely on the larger state department of transportation (DOT) agencies for technical assistance.

Planning

The decision to pave versus maintain an aggregate surface is generally a function of maintenance cost, safety, environmental considerations such as dust and erosion, and future economic development potential. Local politics also plays a role, but often with checks and balances provided by limited resources and multimember boards. Increasing demands for road improvements and limited funds require that engineers effectively plan where and how pavement investments are made.

Design

Several design methods are available for bituminous pavements, including the Asphalt Institute method, *AASHTO Guide for Design of Pavement Structures* (*1*), and state DOT methods. Often, low-volume road agencies have no choice of methods if state or federal funds are used. However, even in such cases, the low-volume road agency can project traffic estimates, select the type of pavement, estimate soil parameters, and have a major influence on the outcome.

Critical Design Considerations

Critical design considerations include selection of pavement design life, aggregate or bituminous base, and pavement drainage.

Pavement Design Life

Several critical design considerations are unique to low-volume roads. Low-volume roads often fail due to age (oxidation and stripping of asphalt cement). At other times, infrequent but heavy loads cause failure. Emphasis solely on equivalent axle loads will result in a short pavement life. An example that comes to mind is paving around an elementary school where the school playground, automobile parking lot, and bus lanes are paved at the same time. Several years later, all require rehabilitation due to cracking and raveling despite a large number of equivalent axle loads projected to remain in the playground and parking lot. It is evident that deterioration from weather and age can be as significant as that from traffic loads.

Design should focus on 15 to 20 years into the future. Design is then a compromise between the current, most economic design and the likely future need. Generally, this indicates that more asphalt is justified to withstand aging effects. The additional asphalt cement will provide increased durability and a reservoir to compensate for oxidation and stripping. The county has also noted that on higher-volume roads, alligator cracking seldom occurs in rutted and bleeding sections high in asphalt content.

Full-Depth Bituminous Designs

Full-depth bituminous designs have not worked well in Blue Earth County. Deterioration with age, apparent softening of the subgrade from capillary water in the fine clay soils, and difficulty of repair have discouraged their use.

Improved Pavement Drainage

Improved pavement drainage appears essential in the frost belt with fine-grained clay soils. Agricultural-type slotted, corrugated drain tile and open-graded aggregate drainage layers have been found to be economical and very effective. A modified aggregate base design where the aggregate base is not carried out to the shoulders (especially for wide shoulders) can result in significant cost savings of one-third or more of the aggregate for a thick aggregate base design with wide shoulders. When combined with an open-graded base and tile system, the design appears to work well. Figure 1 shows a modified aggregate base design. Two projects were constructed with the design 3 and 4 years ago. Both appear to be performing well. They have shown significant flows from the tile systems, especially during the critical spring thaw. There is some concern that water will be trapped in a "bathtub design" and not drain to the ditch inslope. However, as shown below, a dense-graded aggregate base with greater than 10 percent fines passing the No. 200 screen is essentially impermeable, especially when the fines are a clay or silt-type material (*1*).

	Coefficient of Permeability (ft/day) by Percent Passing No. 200 Sieve			
Type of Fines	*0*	*5*	*10*	*15*
Silica or limestone	10	0.07	0.08	0.03
Silt	10	0.08	0.001	0.0002
Clay	10	0.01	0.0005	0.00009

The full aggregate base section under the shoulder, therefore, is not justified for drainage reasons.

Bituminous Pavement Design Variables

During the bituminous pavement design process, the chief variables are the asphalt cement, aggregate, and design method used. Each has major implications for the performance of low-volume bituminous pavements in cold climates.

Asphalt Cement

Asphalt cement is essentially a cheap, largely unprocessed material costing about $100 to $150 per ton or about 5 cents per pound. It has a number of characteristics affecting pavement performance, including (*a*) consistency measured by penetration or viscosity and its variation with temperature and time, (*b*) ductility as new and aged material, and (*c*) pureness and "cracking" damage in the refining process. These characteristics vary due to different sources, and the same source with time.

In Minnesota, as is generally true, the local agency controls only the hardness of asphalt material by specifying penetration. The other variables are ignored, and changes in the resulting performance of pavement are accepted as a risk. It is hoped that the work of SHRP will improve the understanding of this material and refine its use.

Aggregate Properties

A number of aggregate properties are critical to a quality pavement, including gradation of material by size, hardness or toughness, shape, solubility, and affinity for asphalt. Unfortunately, economics dictates use of locally available, inexpensive materials. At $5 to $10 per ton, or about 5 cents per pound, extensive transportation or processing is not feasible.

Available aggregate resources can best be used by optimizing the design of subgrade, base, and pavement layer types and thicknesses. Optimum gradations can be developed by requiring two or more stockpiles graded by size. Crushing can be specified to improve the shape of the aggregate. Coarser materials and increased crushing content can be used when higher strengths are needed. It is also essential to limit the use of soft, deleterious materials to prevent surface spalling and reduced long-term stability.

Mix Design Methods

Generally, the state DOT determines the mix design method for the local agency. It is important to keep in mind the substantial differences between the laboratory and field environments. Small quantities of material are used in the laboratory, that is, a few pounds of materials. In the field, quantities are typically in the hundreds or thousands of tons. Ensuring a uniform, representative sample is difficult at best. In addition, determination of laboratory and field voids depends on compaction. The relationship of compaction energy and method in the laboratory to that in the field is also problematic. Nonetheless, the laboratory trial mix is essential for a satisfactory design to begin production. The laboratory tests provide advance information that either is never available from the field or only becomes available after a significant quantity of material is produced.

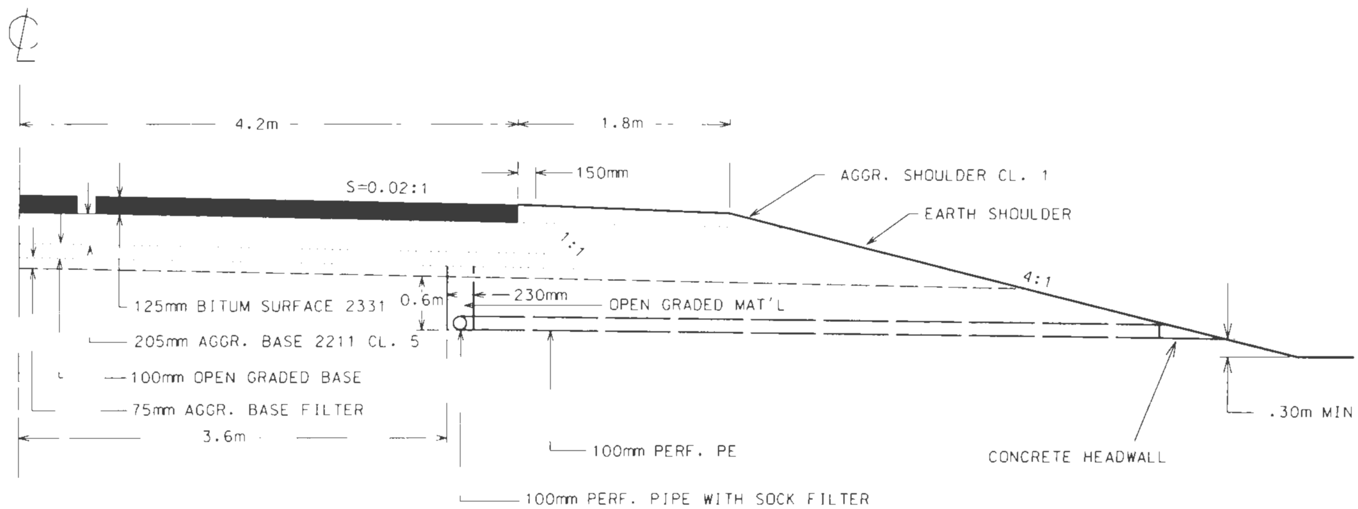


FIGURE 1 Modified aggregate base section.

Design Generalizations for Low-Volume Roads

Air Voids in the Mix

Several studies have shown that appropriate air void content in the mix is essential to a successful pavement (*2–4*). A number of state DOTs, including Minnesota and Colorado, have developed specifications based on this concept. Studies and experience show that high air voids promote deterioration of the pavement from oxidation and stripping. However, low voids promote rutting, bleeding, and flushing. It appears that the lowest voids that will avoid rutting and bleeding will result in the most long-lasting pavement. It then follows that low-volume roads, especially with lighter loads, should tend toward the low end of the acceptable air voids range to increase durability.

Aggregate Quality and Gradation

Hard, durable aggregate that will resist the effects of loads and weather is needed to transfer loads and reduce surface spalling. In the Blue County area, the Los Angeles rattler test is used to specify hardness. However, locally available crushed limestone is frequently used to meet the gradation specification. This material can be soft and susceptible to weathering. Aggregate quality specifications should apply to each aggregate source rock as well as to the entire mixture, or a poor-quality material may result in future pavement failure.

Crushed materials substantially increase stability and reduce rutting. However, as aggregate sources deplete, the larger particles needed for crushing may be unavailable. Use of higher-quality imported material in surface layers is one alternative.

Control of the gradation of materials can be achieved by requiring two or more stockpiles of materials. Proper stockpiling procedures are necessary to avoid segregation. This flexibility to adjust gradation is essential to account for differences in the laboratory and field environments and to implement the integrated design and construction method discussed later in this paper. Perhaps the best measure of the coarseness of the material and gradation is a comparison of the gradation to the maximum density curve as shown in Figure 2.

Low-volume road mixes should tend toward coarser mixes since these mixes can hold more asphalt for a given air void content and should therefore be more durable. In addition, the stability of these mixes is less sensitive to changes in asphalt and air void content as shown in Figure 3.

Asphalt Cement

Selection of the appropriate consistency (as measured by penetration or viscosity) is generally the only control the low-volume road agency has over the asphalt. Other essential characteristics—such as ductility, solubility, and aging—must be accepted as risk at this time. It is important that adequate asphalt be included for present as well as future needs. Generally, low-volume roads should tend toward high asphalt content to ensure durability. Most low-volume road agencies have extensive histories of working with local aggregate sources and mix designs for these sources. Trial mixes or contractor proposals for low asphalt content relative to the history of successful mixes should automatically be suspect. Generally, it is believed that softer asphalt should be chosen for low-volume roads with the hope that it will result in improved ductility and reduced cracking. The

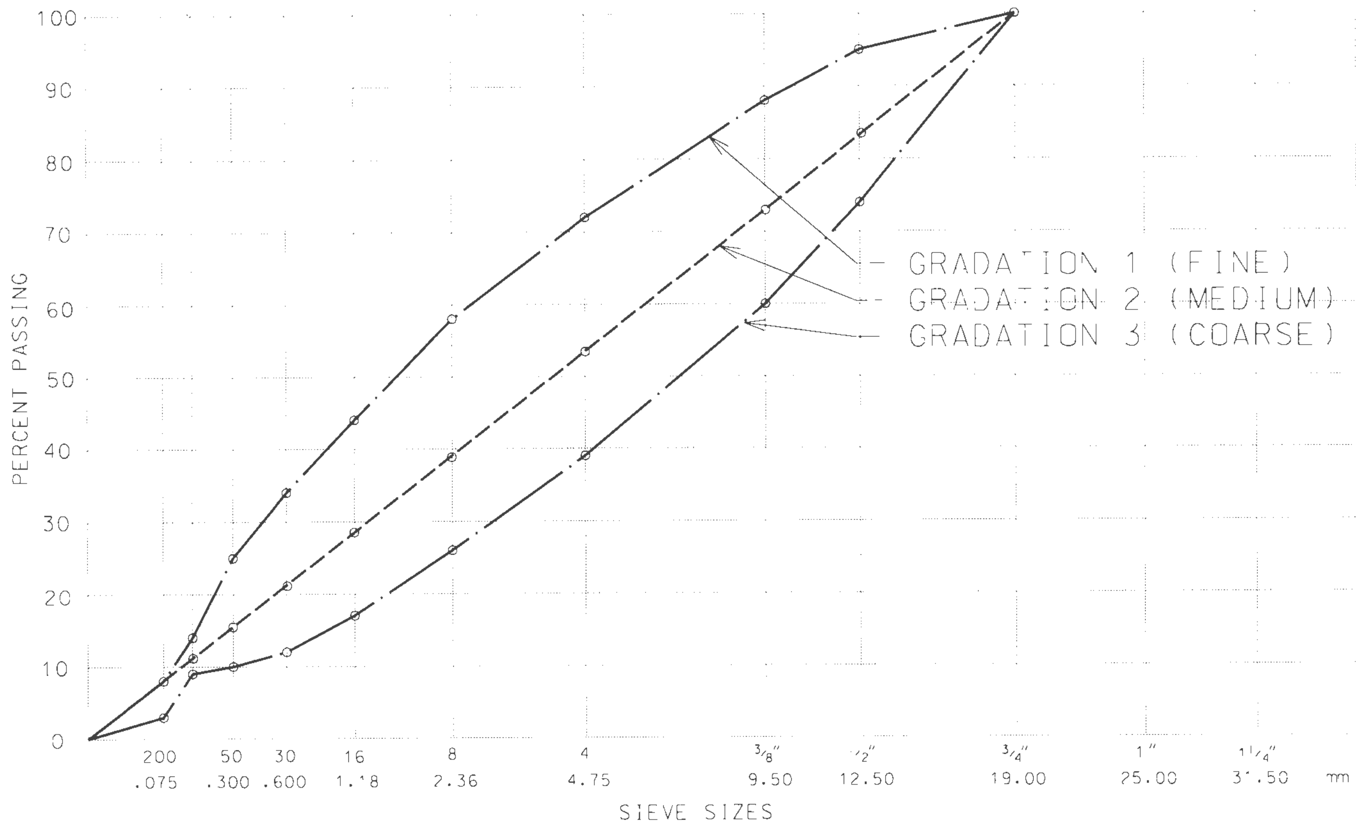


FIGURE 2 Fine, medium, and coarse gradations shown on maximum density plot (*5*).

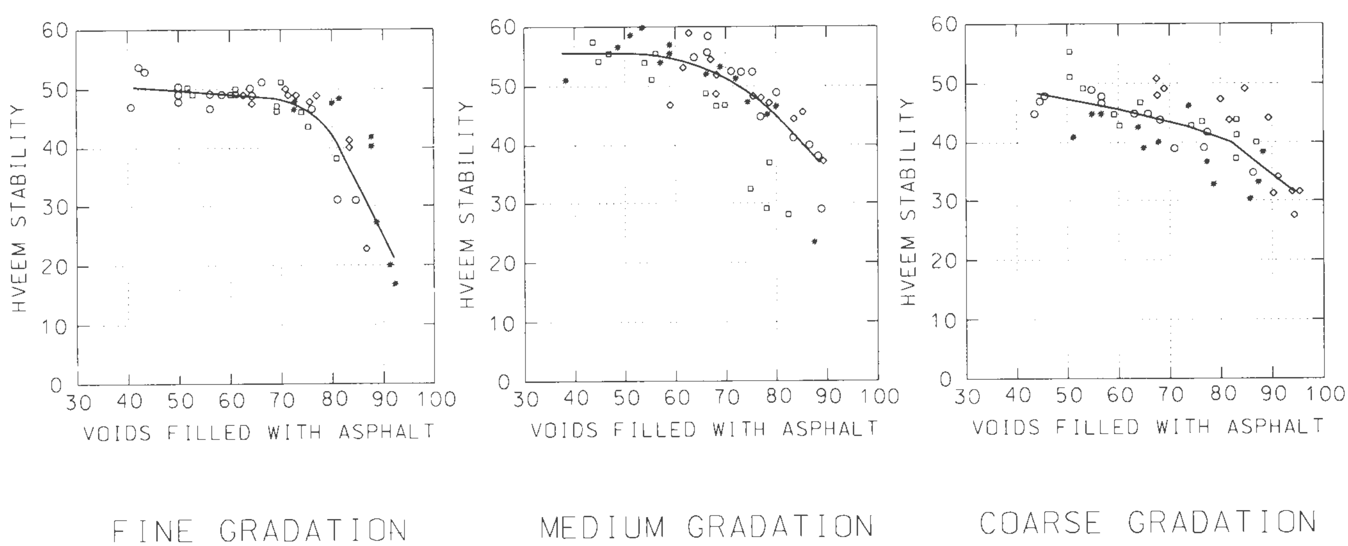


FIGURE 3 Stability versus voids filled with asphalt (*5*). Data shown are from 100 percent crushed material.

exceptional performance of some of the old road mix designs lends credence to this theory.

Construction

As discussed earlier, the laboratory design process is an essential but limited model of the field environment. The real need is to control the air voids, aggregate, and asphalt in the actual product. Conscientious, high-quality contractors and inspectors are needed.

The "tough enforcer" model of the agency that has a constant adversarial relationship with the contractor is giving way to the "partnering" relationship, where early coordination and good communication resolve problems before they occur. It would seem that both models are needed. Early coordination, good communication, and a close working relationship are needed for a quality product. However, constant vigilance and the will to enforce contracts are also needed when the agency and contractor do not always have the same objectives. The will to shut down contractors when necessary and to require that substandard materials be replaced remains essential to consistently procuring a quality product.

Integrated Design and Construction Model

The integrated design and construction model for low-volume bituminous pavements described here attempts to take the design from the bituminous laboratory and complete it in the field. This model includes the following components.

Final Pavement Grade

Good ride quality and consistent structural strength require a uniform pavement section. Yet cores from constructed pavements show a significant difference in thickness. Apparently, the "ski" on the paver has been doing a good job of providing an adequate pavement alignment by varying the thickness of the bituminous layer. This results in a nonuniform pavement with some overly strong and some weak areas. The problem seems to have grown with the retirement of many of the older motor grader operators with their skill for trimming the aggregate base before paving. Fortunately, technology now allows economical automatic control of the vertical alignment by laser or string-line controls.

Stringently Enforced Trial Mix

Production of bituminous material starts by stringently enforcing the trial mix design. The agency should require that all trial mix data and curves be provided. Aggregate gradation and air void samples are taken and asphalt content is measured as soon as production begins.

Adjustment of Asphalt Content

Frequently, the air void content will drift from the design level established for the mix (lower voids range for low traffic volumes). The asphalt content should be adjusted if such an adjustment will provide a void content appropriate for the road (higher asphalt range for low-volume road). If the asphalt content is not adjusted, the gradation should be adjusted.

Adjustment of Aggregate Gradation

The aggregate gradation should be adjusted by examining the relationship between the aggregate gradation and the maximum density curve.

No significant changes in either the asphalt content or gradation should be made without reviewing the stability curves from the trial mix to ensure that stability is not reduced below specifications. Figure 3 shows that stability can decline rapidly after a substantial portion of the voids are filled with asphalt, especially for mixes with fine gradations.

The contract must allow the agency to adjust the asphalt content and aggregate gradation upon request. This requires multiple aggregate stockpiles. It also requires separate pay items for asphalt and aggregate.

Maximum Density Curve Study by Colorado DOT

A Colorado DOT study (*5*) examined 101 mix designs for relationships between air voids and several alternative maximum density plots. It also examined 24 aggregate samples for voids and density relationships for additional variables: (*a*) gradation, (*b*) quantity finer than the No. 200 screen, (*c*) size distribution finer than the No. 200 screen, and (*d*) angularity of fine aggregate. Figure 4 shows alternative maximum density curves considered.

The study found the best relationship between voids and the distance of the gradation from the maximum density line with (*a*) a maximum density curve drawn from the origin to the actual percentage passing the nominal maximum aggregate size and (*b*) the portion of the maximum density line finer than the No. 8 screen. The study also found that the angularity of the particles has an effect on voids.

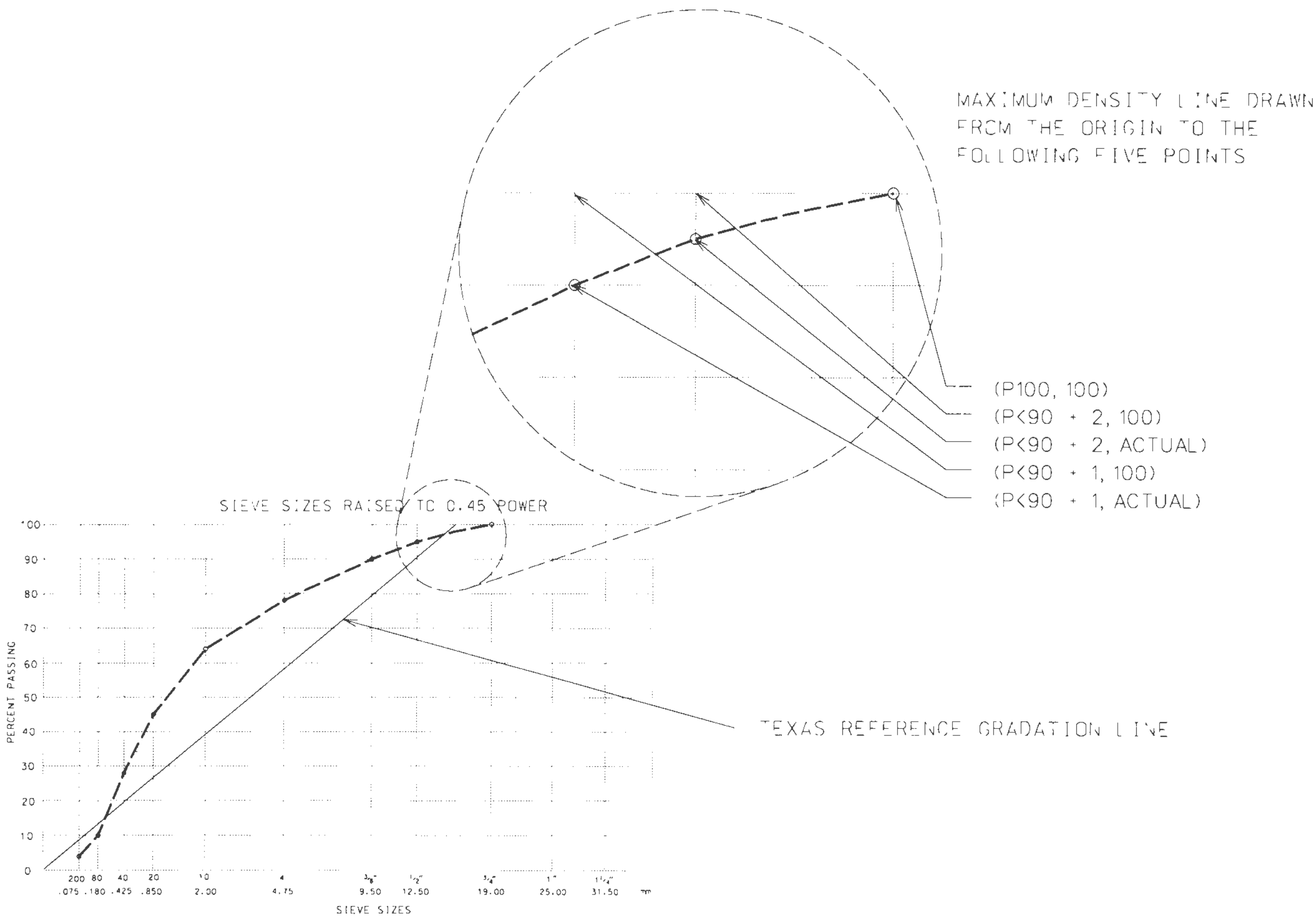


FIGURE 4 Variations of maximum density curves evaluated by Colorado DOT (5).

The study found that the maximum density line was a useful rule of thumb to determine how to adjust gradation to optimize the mixture's air voids. If higher air voids are desired, the gradation is adjusted away from the maximum density line. Conversely, if lower air voids are desired, the gradation is adjusted toward the maximum density line. If adjustment is above the line, the mix is finer, asphalt film thinner, and durability may be lower. If adjustment is below, the opposite may be true.

As an example of the application of asphalt and gradation adjustment to optimize voids, Blue Earth County had a bituminous paving contract with Loveall Construction in the summer of 1993. Gradation and asphalt content tests showed that the contractor was operating within the specifications. However, tests showed that the air void content at 5 percent was above the optimum by 3 to 4 percent for the low-volume road being paved. An asphalt content of 6.5 percent was considered appropriate based on past experience with this aggregate source. The production gradation was extremely close to the trial mix gradation.

The county elected to adjust the mix gradation since the asphalt content was appropriate. After studying the maximum density curve with trial mix and production gradations (see Figure 5), the county engineers instructed the contractor to decrease the sand in increments of 2 percent. This would move the production gradation toward the maximum density line and reduce the voids. A void content test was conducted after each increment until a voids content of 3.5 percent was reached. Production then continued with air voids at 3.5 percent, asphalt content at 6.5 percent, and a gradation that had been modified but was still within the trial mix band. These variables were considered ideal for this low-volume road with its projected use.

Compaction of Mix

Proper compaction of the mix is critical to the long-term performance of the pavement. It is also critical to the achievement of the voids established for the mix in

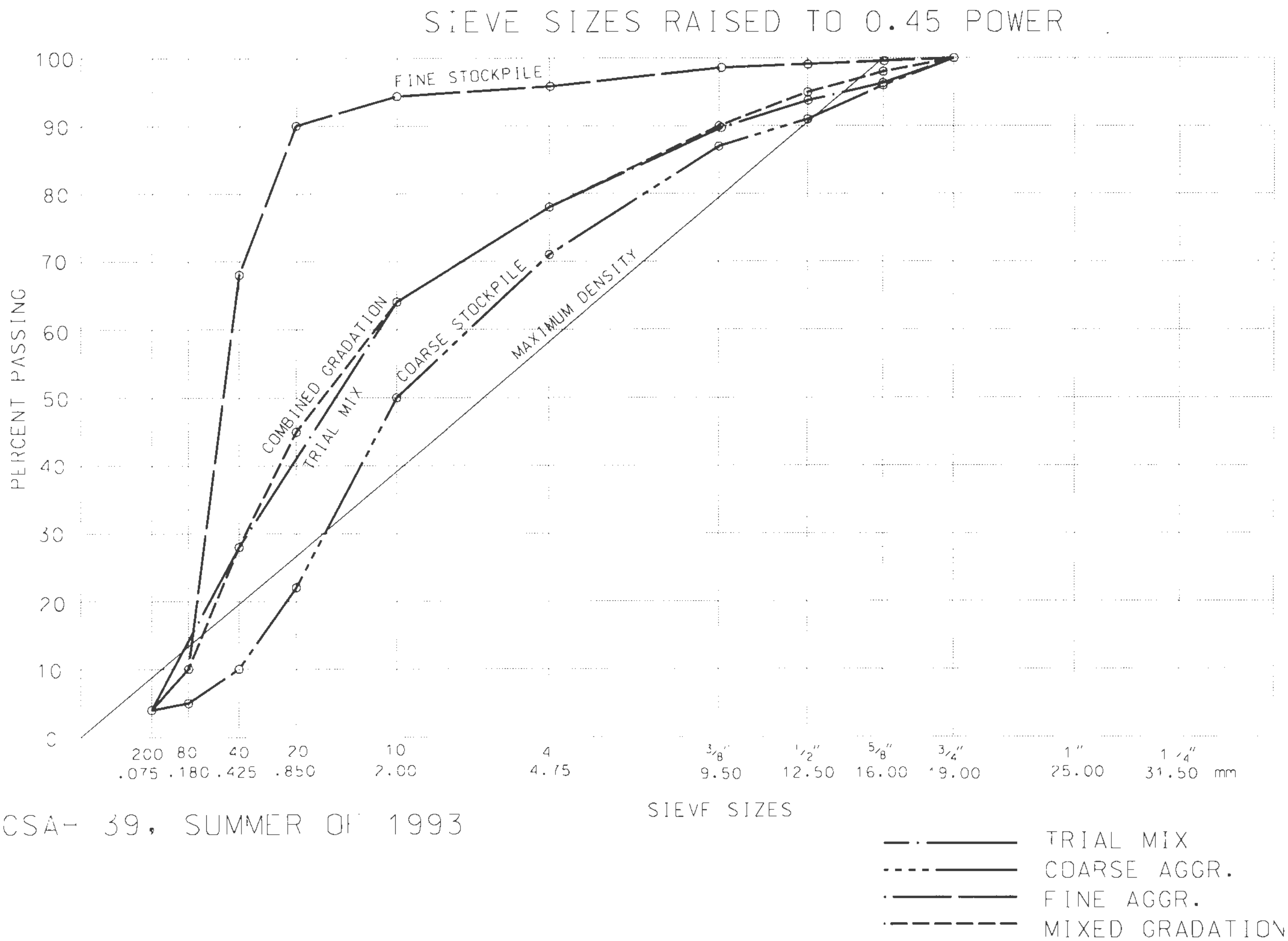


FIGURE 5 Maximum density, trial mix, and production gradation plot.

the laboratory. The contractor is required to construct a control strip to determine the best rolling pattern to reach maximum density. Densities are determined by a portable nuclear gauge furnished by the contractor. The absolute density is interesting but is not as important as the relative maximum density achieved. This reduces the importance of gauge calibration.

The county has been requiring breakdown by a steel roller, intermediate rolling by a rubber-tired roller, and final rolling by a steel roller. Assuming that rolling begins as soon after the paver as possible, this method is somewhat self-enforcing since adequate final rolling is necessary in order to erase the rubber tire marks.

Maintenance

Adequate and appropriate maintenance is an essential element of the quality bituminous pavement system. Pavement management systems should be of tremendous assistance as they are implemented. These systems should supplement experience and expertise of staff and assist in selecting the proper maintenance tool, whether it be crack sealing, seal coating, overlaying, or complete rehabilitation.

Summary

The planning, design, construction, and maintenance of quality bituminous pavements for low-volume roads are still an art but are slowly advancing toward a science. It is hoped that continued research and the efforts of SHRP will help us to better understand and control the many variables and that some of the ideas discussed here, including the integrated design-and-construction concept, will be useful.

REFERENCES

1. *AASHTO Guide for Design of Pavement Structures.* American Association of State Highway and Transportation Officials, Washington, D.C., 1993, pp. 1–19.
2. Heinz, R. E. *Asphalt Concrete Mix Design and Field Control.* FHWA Technical Advisory T 5040.27, 1988, 27 pp.
3. Huber, G. A., and T. S. Shuler. Providing Sufficient Void Space for Asphalt Cement: Relationship of Mineral Aggregate Voids and Aggregate Gradation. In *Effects of Aggregates and Mineral Fillers on Asphalt Mixture Performance.* ASTM STP 1147. ASTM, Philadelphia, Pa., 1992.
4. *Performance Based Specification to Be Used in Mixture Design and Analysis Systems.* Draft Working Paper, SHRP A001. Strategic Highway Research Program, 1991, 51 pp.
5. Aschenbrener, T., and C. MacKean. Factors That Affect the Voids in the Mineral Aggregate of Hot Mix Asphalt. In *Transportation Research Record 1469*, TRB, National Research Council, Washington, D.C., 1994.

Reevaluation of Seal Coating Practices in Minnesota

David W. Janisch, *Minnesota Department of Transportation*

Seal coating of bituminous pavements, referred to as chip sealing in this paper, is a common type of routine maintenance done by local government agencies in Minnesota. Most cities, counties, and rural Minnesota Department of Transportation (Mn/DOT) districts construct at least some seal coats annually. Over the years, Mn/DOT has received calls from local agencies concerned about poorly performing seal coats. This, along with recent developments from the Strategic Highway Research Program (SHRP), led to the development of a seal coat research study. The goal of this study is to find the factors involved in constructing a quality seal coat, including examining the current Mn/DOT specifications and studying the performance of seal coats designed using the procedure found in the Asphalt Institute's asphalt emulsion manual (MS-19), which was used by SHRP. In all, eight local agencies participated in this study: five municipalities and three counties. The test sections were constructed during the summer and fall of 1993. Experiment variables include application rate, sweeping time, aggregate type, and gradation and binder type. These sections will be monitored over the next several years to evaluate their performance. This paper presents an overview of the study, examines the preliminary data, and summarizes the findings. This study will likely lead to changes in the current Mn/DOT bituminous seal coat specification.

The Minnesota Department of Transportation (Mn/DOT) specification for bituminous seal coating (Specification 2356) is found in the 1988 edition of *Standard Specifications for Construction* (*1*). It states that the aggregate shall be spread "at the rate of one pound per square yard for each 0.01 gallon of bituminous material applied" (13.1 kg/m^2 for each liter of bituminous material). This aggregate application rate has been contained in every edition of the standard specifications since 1959. The amount of bituminous material required is outlined in the Mn/DOT *Bituminous Manual* (*2*) and is based on the average particle diameter of the aggregate. The specification does not adjust the application rate to account for the gradation, shape, or specific gravity of the aggregate. Further compounding the problem is the fact that many agencies skip the design procedure altogether and simply assume application rates based on the specified aggregate size and experience.

In contrast, recent chip seals constructed by SHRP (*3*) required the use of the design procedure contained in the Asphalt Institute's MS-19, 1979 edition (*4*). This design procedure was developed by McLeod in the 1960s. It is outlined in proceedings from the 1960 and 1969 annual meetings of the Association of Asphalt Paving Technologists (*5,6*). This procedure is called the McLeod procedure for the remainder of this paper.

Over 160 km (100 mi) of pavement was chip-sealed as part of this study. Five agencies constructed chip seals using both their standard application rates and application rates determined by the McLeod procedure. Test sections were also constructed using various aggregates (granite, trap rock, limestone, pea rock), binders (CRS-2, CRS-2P, HFMS-2, RC 800), construction techniques (standard seal and choke seal), and curing times (early and late sweeping).

Mn/DOT Design Procedure

The Mn/DOT design procedure is based on a measurement termed the average particle diameter (APD), sometimes called the spread modulus. The APD provides a measure of the average seal coat thickness. It is defined as the weighted average of the mean size in millimeters of the largest 20 percent, the middle 60 percent, and the smallest 20 percent of the aggregate particles. The mean size is determined by projecting a vertical line from the 10 percent, 50 percent, and 90 percent passing line. The APD is then determined using the following equation:

$$
\begin{aligned}
APD &= (0.20)(90\%\ \text{passing size}) \\
&\quad + (0.60)(50\%\ \text{passing size}) \\
&\quad + (0.20)(10\%\ \text{passing size})
\end{aligned} \tag{1}
$$

Once the APD is known, the binder application rate is determined by using one of the following equations:

For cutbacks and asphalt emulsions:

$$
\text{Binder application rate} = \begin{cases} (\text{L/m}^2) \\ (\text{gal/yd}^2) \end{cases} = \begin{cases} (0.177)(APD,\ \text{mm}) \\ (1.0)(APD,\ \text{in.}) \end{cases} \tag{2}
$$

For asphalt cements:

$$
\text{Binder application rate} = \begin{cases} (\text{L/m}^2) \\ (\text{gal/yd}^2) \end{cases} = \begin{cases} (0.124)(APD,\ \text{mm}) \\ (0.7)(APD,\ \text{in.}) \end{cases} \tag{3}
$$

For example, suppose we are using an aggregate that has the gradation shown in Figure 1. The mean sizes of the largest 20 percent, middle 60 percent and smallest 20 percent are 9.0 mm (0.354 in.), 6.3 mm (0.248 in.), and 3.5 mm (0.138 in.), respectively. From Equation 1,

$$
\begin{aligned}
APD &= (0.20)(9.0\ \text{mm}) + (0.60)(6.3\ \text{mm}) \\
&\quad + (0.20)(3.5\ \text{mm}) = 6.28\ \text{mm}\ (0.247\ \text{in.})
\end{aligned} \tag{4}
$$

Assuming that an asphalt emulsion is to be used, the binder application rate is determined using Equation 2:

$$
\text{Binder application rate} = \begin{cases} (0.177)(6.28\ \text{mm}) \\ (1.0)(0.247\ \text{in.}) \end{cases} = \begin{cases} 1.11\text{L/m}^2 \\ 0.247(0.25)\ \text{gal/yd}^2 \end{cases} \tag{5}
$$

As mentioned earlier, Mn/DOT specifications state that the aggregate shall be spread at the rate of 1 lb/yd^2 for each 0.01 gal of bituminous material applied (13.1 kg/m^2 for each liter of bituminous material). This results in the following aggregate application rates:

Aggregate application rate =

$$
\begin{cases} (1.11\ \text{L/m}^2) \times \left(\dfrac{13.1\ \text{kg/m}^2}{1\ \text{L/m}^2}\right) = 14.5\ \text{kg/m}^2 \\ (0.25\ \text{gal/yd}^2) \times \left(\dfrac{1\ \text{lb/yd}^2}{0.01\ \text{gal/yd}^2}\right) = 25\ \text{lb/yd}^2 \end{cases} \tag{6}
$$

McLeod Design Procedure

The McLeod procedure also determines the aggregate and binder application rates. Aggregate application rates depend on gradation, shape (measured by the flakiness index), and specific gravity. Binder application rates depend on the aggregate gradation and shape, traffic volume, existing pavement condition, and binder properties. The procedure is based on the following factors:

1. There is a certain amount of a given aggregate that can be spread one stone thick over 1 m^2 of pavement.
2. The voids in this aggregate layer need to be 70 percent filled with asphalt binder for good performance.

Some key components of the design procedure are

- Loose unit weight of the cover aggregate,
- Voids in the cover aggregate in a loose condition,
- Flakiness index,
- Mean aggregate size, and
- Average least dimension.

The loose unit weight of the cover aggregate is determined according to ASTM C 29 and is needed to calculate the voids in the aggregate in a loose condition. There was a wide range of loose unit weights for the samples from this study due to the different gradations and aggregate types. Average loose unit weights were

90, 95, and 100 lb/ft^3 for granite, trap rock, and pea rock, respectively. The loose unit weight depends more on the gradation of the aggregate than it does on specific gravity.

The voids in the cover aggregate in a loose condition approximates the voids present when the chips are dropped from the spreader onto the pavement. This value will be near 50 percent for a one-sized aggregate (4), less for graded aggregate. The voids in the samples from this study averaged 45 percent and ranged from 37 to 50 percent. After initial rolling, the voids are assumed to be reduced to 30 percent, and finally to 20 percent after sufficient traffic has oriented the stones on their flattest side.

The flakiness index is a measure of the percentage by weight of flat particles. It involves testing a small sample of aggregate particles for their ability to fit through a slotted plate. They will fit through the plate if they have a flat side smaller than 70 percent of the sieve opening on which they were retained. For example, any chip retained on the 12.5-mm (0.5-in.) sieve that has a flat side thinner than 8.75 mm (0.35 in.) will pass through the plate opening. The plate contains slots for material retained on the 19.0, 12.5, 9.5, 6.3, and 4.75 mm (3/4 in., 1/2 in., 3/8 in., 1/4 in., and No. 4) sieves.

The median aggregate size is determined from the gradation chart. It is the theoretical sieve size through which 50 percent of the material passes. The median aggregate size represents the mean thickness of the seal coat and is found by projecting a vertical line at the 50 percent passing size (see Figure 1).

The average least dimension is determined from the median aggregate size and the flakiness index. It is a reduction of the median aggregate size after accounting for flat particles.

Samples of the aggregate and binder were submitted to Mn/DOT's Materials Research and Engineering Laboratory for testing. The aggregates were tested for gradation, bulk specific gravity, loose unit weight, and flakiness index determination. The binder was tested for compliance with specifications and determination of the residual asphalt content.

The aggregate application rate (C) is determined from the following equations:

$$C = \begin{cases} (1 - 0.4V)HGE \\ 46.8(1-0.4V)HGE \end{cases} \tag{7}$$

where

C = cover aggregate application rate (kg/m^2) (lb/yd^2),
V = voids in the loose aggregate, in percentage expressed as a decimal:

$$V = \begin{cases} 1 - \dfrac{W}{1000G} \\ 1 - \dfrac{W}{62.4G} \end{cases} \tag{8}$$

H = average least dimension (mm) (in.),
G = bulk specific gravity of the aggregate,
E = wastage factor for traffic whip-off (ex: 1.10 for 10 percent wastage),
W = loose unit weight of the cover aggregate, ASTM Method C 29 (kg/m^3) (lb/ft^3).

For example, assume that the aggregate used in the previous example (Figure 1) also has the properties given in Table 1. Using these values and Equations 7 and 8, the aggregate application rate is calculated as follows:

$$C = \begin{cases} [1 - (0.4)(0.48)] \times (4.7\text{ mm}) \times (2.71) \\ \times (1.10) = 11.3\ (11\text{ kg/m}^2) \\ (46.8) \times [1 - (0.4)(0.48)] \times (0.185\text{ in.}) \\ \times (2.71) \times (1.10) = 20.8\ (21\text{ lb/yd}^2) \end{cases} \tag{9}$$

The binder application rate (B) depends not only on the properties of the aggregate mentioned above but also on the existing pavement condition, traffic volume, aggregate absorption, and residual asphalt content of

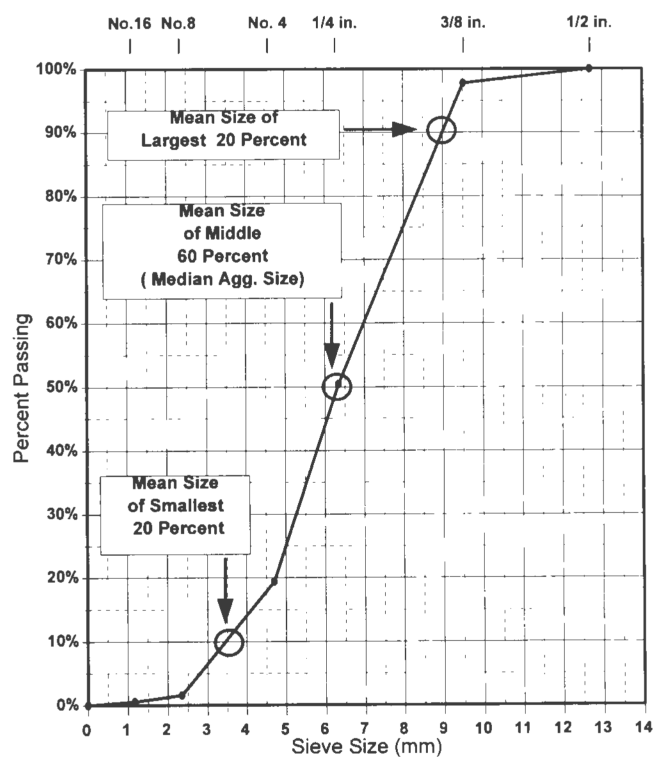


FIGURE 1 Gradation of aggregate in design example (1 mm = 0.039 in.).

TABLE 1 Input for McLeod Aggregate Application Rate Design Example

Type of Test	S.I. Metric Units
Median Particle Size	6.3 mm
Flakiness Index	20.8 percent
Average Least Dimension	4.7 mm
Loose Unit Weight of Aggregate	1,396.50 kg/m³
Voids in the Cover Aggregate	0.48
Bulk Specific Gravity	2.71
Wastage Factor Due to Traffic Whip-Off, E	1.10

1 mm = 0.039 in., 1 kg/m³ = 0.062 lb/ft³

the binder. The binder application rate is determined as follows:

$$B = \begin{cases} \dfrac{(0.40)(H)(T)(V) + S + A}{R} \\ \dfrac{(2.244)(H)(T)(V) + S + A}{R} \end{cases} \qquad (10)$$

where

B = binder application rate (L/m²) (gal/yd²),
H = average least dimension (mm) (in.),
T = traffic factor (based on expected vehicles per day),
V = voids in the loose aggregate, in percentage expressed as a decimal (Equation 8),
S = surface condition factor (L/m²) (gal/yd²) (based on the "dryness" of the existing surface),
A = aggregate absorption factor (L/m²) (gal/yd²) (equal to zero unless aggregate is porous), and
R = residual asphalt content of binder, in percentage expressed as a decimal.

Typical values for determining the binder application rate are shown in Table 2. Using these values, the binder application rate, B, is calculated from Equation 10.

$$\mathrm{B} = \begin{cases} \dfrac{(0.40)(4.7\ \mathrm{mm})(0.70)(0.48) + 0.27\ \mathrm{L/m^2} + 0)}{0.67} = 1.35\ \mathrm{L/m^2} \\ \dfrac{(2.244)(0.185\ \mathrm{in.})(0.70)(0.48) + 0.06\ \mathrm{gal/yd^2} + 0}{0.67} = 0.30\ \mathrm{gal/yd^2} \end{cases} \qquad (11)$$

Comparison of Design Application Rates

For this example, the binder application rate is 1.11 L/m² (0.25 gal/yd²) for the Mn/DOT procedure and 1.35 L/m² (0.30 gal/yd²) for the McLeod procedure. The aggregate application rate is 15 kg/m² (25 lb/yd²) for the Mn/DOT procedure and 11 kg/m² (21 lb/yd²) for the McLeod procedure. This is the common trend found when comparing the two design procedures. Most of the time, the Mn/DOT procedure recommends more aggregate and less binder than the McLeod procedure. A comparison of the aggregate and binder application rates for all 40 samples tested is shown in Figures 2 and 3.

Problems With Current Mn/DOT Design Procedure

Several problems are believed to contribute to the poor performance of seal coats in Minnesota. Among them are the following.

- The Mn/DOT procedure recommends the same binder application rate for all emulsions and cutbacks. A typical RC-800 cutback contains 85 percent residual asphalt compared with only 67 percent for a CRS-2 emulsion. As a result, if these two binders are applied at the same rate, the emulsion will contain 21 percent less asphalt than the cutback once the cutter or water has evaporated. Since the residual asphalt bonds to the stone particles, having the binder application rate based on this residual asphalt content is vital for proper embedment of the aggregate particles.

TABLE 2 Input for McLeod Binder Application Rate Design Example

Type of Test	S.I. Metric Units
Surface Condition Factor, S	0.27 liter/m^2 (slightly pocked, porous and oxidized surface)
Traffic Factor, T	0.70 (500 to 1,000 vehicles per day)
Aggregate Absorption Factor, A	0.0 (disregarded except for obviously porous stone)
Residual Asphalt Content of Binder, R	0.67 (Typical value for CRS-2 in Minnesota)

1 liter/m^2 = 0.22 gallon/yd^2

- The Mn/DOT FA-3 (AASHTO M43, Size No. 8) gradation does not require the 9.5-mm (1/4-in.) sieve. Requirements are given for the 9.5-mm (3/8-in.) and 4.75-mm (No. 4) sieves. This gap in successive sieves sizes (4.75 mm, 0.188 in.) results in large differences in material considered to be the same. For example, one sample of FA-3 material had 100 percent passing the 6.3-mm (1/4-in.) sieve, whereas another had only 30 percent passing. This large difference was not detected using the normal sieve nest and will lead to problems when agencies use the same application rates from year to year.
- The Mn/DOT procedure recommends the same amount of cover stone for all aggregate types and gradations as long as the average particle diameter is the same. This is a problem because a given mass of trap rock will not cover as large an area as the same mass of pea rock because of differences in specific gravity (2.98 for trap rock, 2.66 for pea rock).

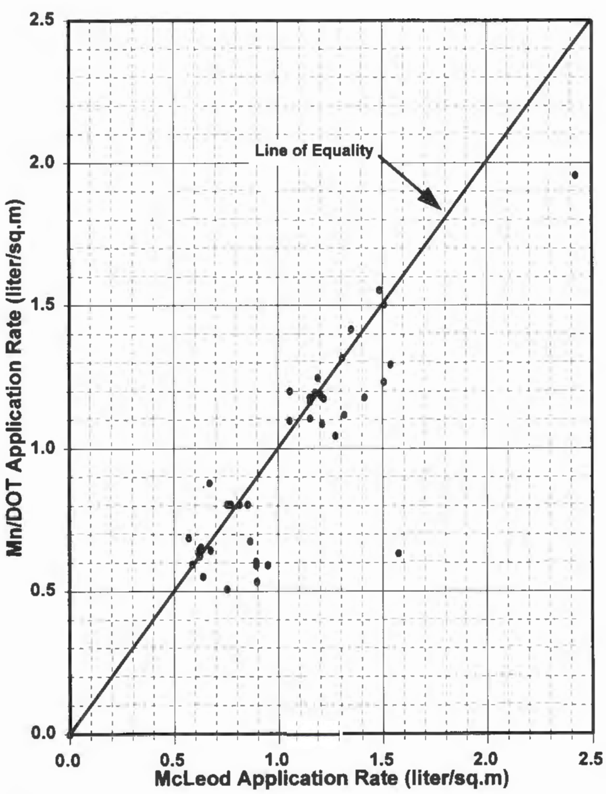


FIGURE 2 Comparison of binder design application rates (1 L/m^2 = 0.22 gal/yd^2).

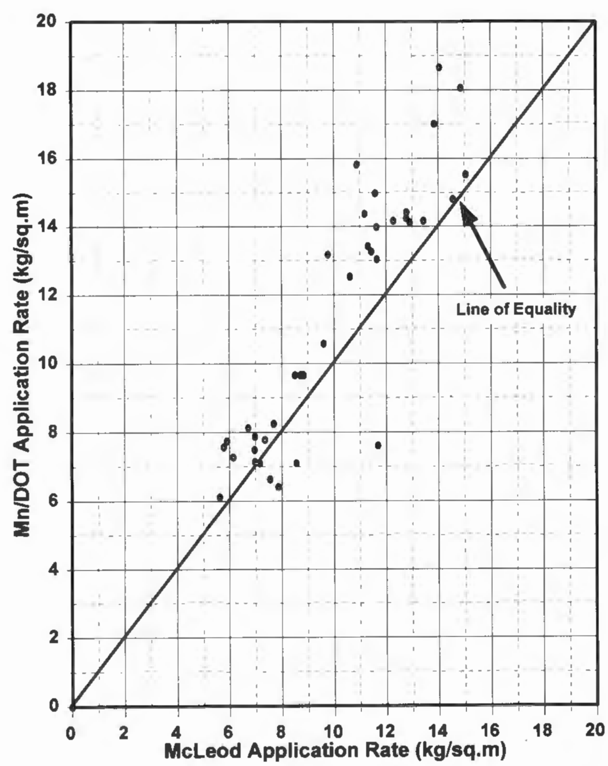


FIGURE 3 Comparison of aggregate design application rates (1 kg/m^2 = 1.84 lb/yd^2).

- The Mn/DOT procedure makes no adjustments for one-sized aggregates. It is quite possible to have two aggregates with the same average particle diameter and very different gradations. An aggregate that is one-sized will have more void space to fill with binder than a graded aggregate.
- No adjustments are made in the Mn/DOT procedure for flat aggregate. Samples from this study ranged from a low of 9 to a high of 36 percent flat particles by weight (flakiness index). It is assumed that over time, traffic will cause the chips to lie on their flattest side. As a result, the chip seal will be thinner when using flat aggregate than it will when using cubical aggregate. To obtain the proper embedment, this thickness and its corresponding void content must be known.
- No adjustments are suggested in the Mn/DOT procedure for adjusting the binder application rate to account for traffic or surface condition other than experience.
- The Mn/DOT procedure usually results in more aggregate and less binder than the McLeod procedure. This combination has the potential for large amounts of premature aggregate loss.
- The Mn/DOT procedure usually results in seal coats with a multiple-stone thickness rather than the desired one-stone thickness. If the large amount of loose stone is not swept soon after it is placed, traffic will cause it to act like an abrasive, grinding off and/or wedging between the stones that are properly embedded. In addition, the surface often has so much loose aggregate, the rolling operation does not orient the stones on their flat side as is needed. This is because the roller is not in contact with the stones that are touching the existing road and embedded in the binder.

Observations Made During Construction

Several observations were made by the author during the construction of the chip seals sections:

- While some agencies calibrated the distributor before construction, no calibration of the aggregate application rate was being done by any of the contractors or agencies. In addition, the same application rate was being applied to all of the sections provided the weather conditions did not change. No corrections in binder application rates were made to adjust for traffic or existing pavement condition.
- The projects that used three rollers did a much better job of achieving full coverage before the emulsion "broke." When two rollers were used, they had difficulty keeping up with the distributor. Usually these rollers were exceeding the specified 8.3 km/hr (5 mi/hr). In one extreme case, the rollers were so far behind the distributor, in an attempt to catch up they were traveling close to 33 km/hr (20 mi/hr), leaving a wake of loose chips in their path.
- The bituminous distributor operator plays a vital role in the success or failure of the project. The difference between experienced and inexperienced operators was obvious. The inexperienced operators had large overlaps, long delays (particularly on culs-de-sac). Some delayed so much that the binder "broke" long before any chips were placed.
- There does not appear to be a standard way to seal-coat culs-de-sac. Nearly every agency and/or contractor sealed them differently. Some sprayed the entire cul-de-sac with binder before spreading any chips; some drove the distributor, spreader, and roller in circles; some started at the far end of the cul-de-sac, while others started at the radius.

Current Condition of Test Sections

Most of the sites were visited in February and November of 1994 to find out how well the sections held up to snowplow blades over the winter and traffic over the summer. There is little or no damage from plow blades on any of the sections to date. In addition, both the designed and undesigned sections appear to have a very high degree of chip retention despite aggregate type or size. However, the undesigned sections have a much more irregular-looking surface than the designed sections. A more in-depth condition assessment will be made in subsequent years.

Conclusion

Since the projects described in this paper were constructed in 1993, there are no long-term performance data. However, several conclusions are felt to be appropriate at this time.

- Very few of the agencies run gradations before seal-coating. As a result, no design procedure is used. Depending on the specified size (FA-3, FA-2, etc.), the binder rates are chosen based on experience. The aggregate is then applied at the specified rate of 1 lb/yd^2 for each 0.01 gallon of bituminous material applied (13.1 kg/m^2 for each liter of bituminous material). This often results in as much as 16 to 19 kg/m^2 (30 to 35 lb/yd^2) of aggregate.
- Aggregate application rates were reduced by as much as 50 percent when using the McLeod design procedure instead of the agencies' standard rate. The McLeod procedure always reduced the amount of ag-

gregate recommended when compared to the agencies' standard.

- Usually, the Mn/DOT design procedure recommended more aggregate and less binder than the McLeod procedure.
- Sweeping time was significantly reduced when using the design application rates rather than the agencies' standard rate. This is due to the designed seal coats being only one stone thick. As a result, there is very little loose, nonembedded aggregate to sweep up.
- To date, seal coats designed using the McLeod procedures perform as well as or better than undesigned seal coats.

Recommendations

- Mn/DOT's current seal coat aggregate gradation requirements should include the 6.3-mm sieve (U.S. No. 3, 0.25 in.) in the nest to better characterize the gradation of FA-3 material. This will provide for a more uniform product from year to year.
- Aggregate samples submitted for design should be taken from several areas of the stockpile after it is on the job site rather than submitted from the source pit due to considerable variability in the material.
- Calibration of the equipment, particularly the chip spreader, is crucial, easy to do, and should be required as part of the specification. Calibration of the chip spreader should be done whenever the design application rate changes. The ASTM draft method for chip spreader calibration is recommended. This procedure involves placing 10 to 12 one-foot-wide (30.5-cm) ribbed rubber mats side by side and driving the spreader over them as it drops chips. The longitudinal spread rate is then determined by weighing the amount of aggregate retained on each mat. The transverse spread rate is determined by comparing the amount of stone on each of the mats. Adjustments are then made to the gate openings so they apply a uniform spread rate.
- Sweeping should occur as soon as possible after construction, normally the day after sealing. Leaving loose stones on the roadway is dangerous and is believed to be detrimental to seal coat life.
- The Minnesota DOT should continue to monitor the performance of these sections and modify the existing seal coat specification (2356) and *Bituminous Manual* accordingly.

References

1. *Standard Specifications for Construction.* Minnesota Department of Transportation, 1988.
2. *Bituminous Manual.* Section 5-693.600 Minnesota Department of Transportation, 1990.
3. Bullard, D. J., R. E. Smith, and T. J. Freeman. Development of a Procedure to Rate the Application of Pavement Maintenance Treatment. *Strategic Highway Research Program.* Report No. SHRP-M/FR-92-102. 1992.
4. *A Basic Asphalt Emulsion Manual.* Manual Series No. 19 (MS-19). Asphalt Institute, 1979.
5. McLeod, N. W. Basic Principles for the Design and Construction of Seal Coats and Surface Treatments With Cutback Asphalt and Asphalt Cements. *Proc., Association of Asphalt Paving Technologists*, Supplement to Vol. 29, 1960.
6. McLeod, N. W. A General Method of Design for Seal Coats and Surface Treatments. *Proc., Association of Asphalt Paving Technologists*, Vol. 38, 1969.

Effects of Vehicle Type and Tire Pressure on Dry-Weather Road Maintenance

Paul H. Greenfield, *USDA Forest Service, San Dimas, California*
S. Colin Ashmore, *Nevada Automotive Test Center*

In 1993 a structured test was conducted to measure the effects of different vehicle types on road maintenance for operation in a dry climate. Three different vehicle types were compared: high-tire-pressure logging truck, low-tire-pressure logging truck (both were 18-wheel, western U.S.–style logging trucks), and a mix of light, two-axle vehicles. The test track consisted of four different sections and three individual lanes. Comparisons were made for roadway roughness (washboarding, potholing), material loss, and rutting. Results are presented indicating that grade and alignment have substantial impacts on road maintenance ratios between light and heavy vehicles operating under dry conditions. Maintenance ratios between light and heavy vehicles are on the order of five to one for the dry operating conditions experienced.

A number of studies have been performed that have attempted to quantify the effects of different vehicles on paved surfaces. Gibby et al. (*1*) review the rationale for allocating maintenance costs among various users. Their study resulted in about a 1 to 95 damage ratio between light and heavy vehicles. Relatively little is known about the maintenance effects of different vehicles on unpaved low-volume roads. The World Bank's HDM-III model is perhaps one of the best and most commonly accepted models, but it does not specifically address road maintenance concerns for operation in a dry environment where roughness is dependent on the vehicle's characteristics (*2*). Paige-Green and Visser (*3*) use an algorithm to predict roughness, but vehicle type is not specifically handled.

In 1993, a test was conducted to determine the maintenance effects of vehicles in dry-weather conditions. The test was designed to examine failures that occur not from shear failure of the road surface but from the mechanistic failure that leads to road roughness in the form of roadway corrugations (washboarding), potholes, and surface material loss. This study was undertaken because the proponents felt that currently accepted ratios for maintenance of aggregate and native roadways do not always address these failure modes for dry-weather operation and place too much emphasis on the heavy vehicle as the major cause of road deterioration regardless of seasonal operating variables.

Objectives

The U.S. Department of Agriculture (USDA) Forest Service Commensurate Share Study was conducted at the Nevada Automotive Test Center (NATC) in Carson City, Nevada. It was designed to quantify and compare the effects on road deterioration from the operation of heavy and light vehicles in dry-weather conditions (*4*). The objective was to determine road-user maintenance cost share ratios between logging truck traffic with high-pressure tires, logging truck traffic with low-pressure tires [Central Tire Inflation System (CTIS) operation], and light vehicle traffic. These ratios play an important part in determining the relative shares of responsibility for road maintenance where both the government and the private sector have traffic over the same road. Both aggregate and native road surfaces were evaluated in this study.

The tests were designed to achieve the following subsidiary objectives:

- Provide evaluation of vehicle weight and operating characteristics on road-user maintenance;
- Provide evaluation of the causes of maintenance; and
- Provide guidance for the planning and execution of subsequent field tests for local verification.

The test was designed to allow calculation of commensurate share ratios. These ratios are used by the USDA Forest Service to determine relative responsibilities for road maintenance, and they are based on the amounts of traffic attributable to each party. It is important to note that in this test design, all variables that could be controlled were identical for the three test track lanes. All lanes were constructed the same, maintained the same, and measured the same, and all traffic was applied in a similar manner (closely controlled speeds, loads, loaded and unloaded lap percentages, etc.). The number of laps to failure may be different based on local conditions and maintenance practices; however, the ratios should be approximately the same for dry-weather conditions.

Test Sections

Four test sections were constructed that were representative of typical Forest Service road design and aggregate material selection. The course shown in Figure 1 was constructed with 76 mm (3 in.) of 38-mm (1.5-in.) minus base material and 15.2 cm (6 in.) of 25-mm (1-in.) minus surfacing material. The native test section consisted of 15.2 cm (6 in.) of compacted native soil. In reference to Figure 1, the loaded vehicles traveled clockwise, and the unloaded vehicles traveled counterclockwise, resulting in a combination of adverse and favorable grades. The track geometry consisted of the following:

- 8 percent grade favorable: an 8 percent aggregate grade favorable to loaded logging truck traffic tangent (straight), 91.5 m (300 ft) long, 3.7 m (12 ft) wide;
- 3 percent grade (adverse): a 3 percent aggregate grade adverse to loaded logging truck traffic tangent (straight), 91.5 m (300 ft) long, 3.7 m (12 ft) wide;
- Curve: 27.5-m (90-ft) radius, 180-degree aggregate curve on a 2 to 3 percent grade with an apex in the middle of the curve equal radii, 86.3 m (283 ft) long, 4.3 to 4.9 m (14 to 16 ft) wide; and
- Native (3 percent adverse): a 3 percent grade with native surfacing material, adverse to loaded logging truck traffic tangent (straight), 91.5 m (300 ft) long, 3.7 m (12 ft) wide.

The aggregate test sections consisted of 76 mm (3 in.) of 38-mm (1.5-in.) minus base material, design California bearing ratio (CBR) values of 20 to 25, and an actual average CBR value of 21.4; and 152 mm (6 in.) of 25-mm (1-in.) minus surfacing material, design CBR value of 60, and actual average CBR value of 40.5. The native test section contained 15.2 cm (6 in.) of compacted material.

Base Material

Although no requirements were specified for the subbase material, it was sampled with the following results. The gradation showed that the subbase material met a C grading according to Forest Service EM-7720-100LL, dated April 1985. The grading analysis (AASHTO T11, T27) was as follows: Atterberg Limits (AASHTO T89), plasticity index, 13; liquid limit, 36. Los Angeles abrasion (AASHTO T96), 500 rev, % loss, 14.7; durability index (AASHTO T210), course, 55; fine, 26.

Maintenance Blading

All maintenance for the test sections was performed with a John Deere 570 grader; the procedure was always the same between the test sections. After the test section failed due to one of the four failure criteria, it was bladed using a "tight blading" technique in which the material was cut with the blade angled (the cutting edge and the blade itself) and then carried and rolled within the blade width, remixing the fines and segregated rock. Before the blading, the moisture content was increased by running a water truck over the failed test section. The actual maintenance blading operation consisted of four to six alternating passes with a water truck and grader, followed by limited wheel compaction with the water truck. The surfacing material was worked back and forth across the lane width by running the grader in opposite directions through the test section.

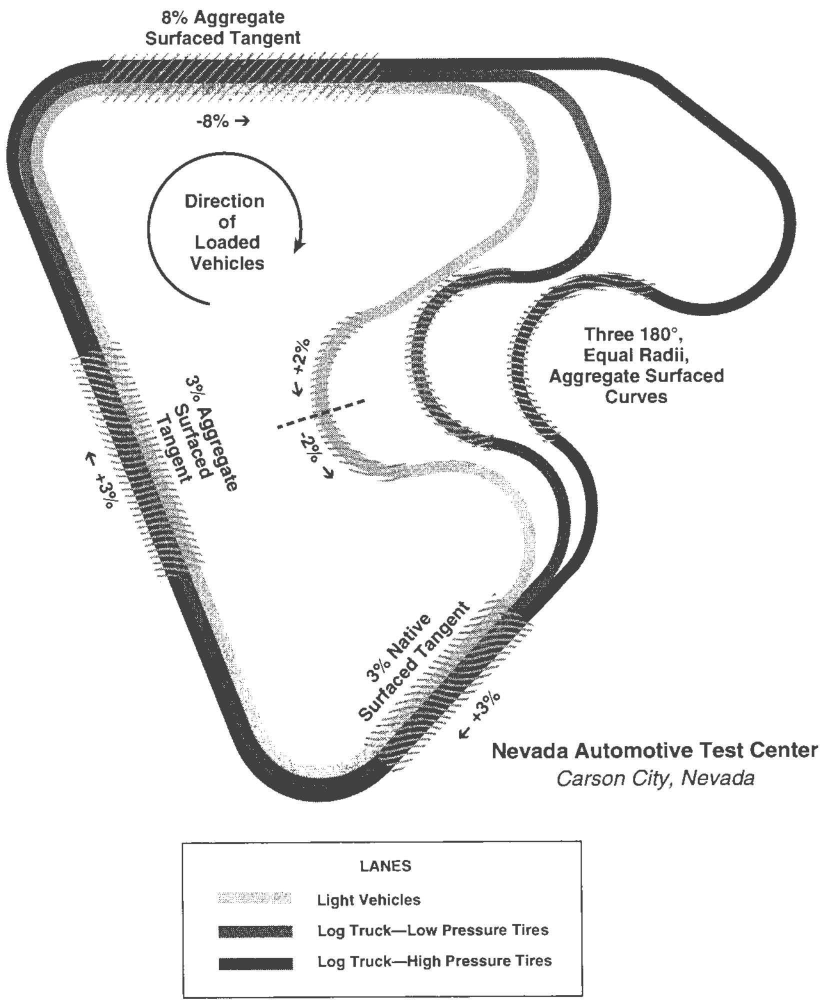


FIGURE 1 Diagram of NATC Test Track for dry-weather maintenance comparisons.

Traffic Lanes

As shown in Figure 1, each test section was three lanes wide, each lane dedicated to a specific vehicle class, and no mixed-vehicle running was conducted. The following were the vehicle classes to which each lane was dedicated:

- Inside lane: light vehicle traffic,
- Middle lane: logging truck traffic with low tire inflation pressures,
- Outside lane: logging truck traffic with high tire inflation pressures.

Vehicle Selection

Four light vehicles (Figure 2) were selected to represent typical traffic using Forest Service roads. Before the start of the test, the vehicles were inspected, serviced, and aligned to factory specifications. The shock absorbers were verified in good condition. New tires were installed, and tire pressures were set to the manufacturer's recommendations for highways. Scheduled services, tire and shock inspections, and maintenance of the vehicles were performed throughout the test.

The target percentage of the total running for the light-vehicle lane was as follows:

Type of Vehicle	*Loaded or Unloaded*	*Percentage of Total Running*	*Direction on 8 Percent Grade*
680 kg (3/4 ton) pickup truck	Loaded	40	Down
680 kg (3/4 ton) pickup truck	Unloaded	40	Up
Light pickup	Unloaded	10	Either
Utility vehicle	Unloaded	10	Either

FIGURE 2 Light vehicles used for comparison testing.

The test design required that the loaded and unloaded 680-kg (3/4-ton) pickups travel in opposite directions around the course. Also, the 680-kg (3/4-ton) pickup traffic represented 80 percent of the total laps in the light-vehicle lane. Given that the unloaded pickups produced rapid road deterioration on the 8 percent grade, these vehicles were run either one for one (i.e., passing in the transition areas) or on an hour rotation between the two trucks (approximately 37 passes per hour).

Logging Trucks

Two nearly identical Class 8 western U.S.–style logging trucks were selected to meet the contract requirements for two heavy logging trucks (Figure 3). These vehicles had five axles—one steering axle, two drive axles, and two trailer axles—in the loaded configuration, and one steering and two drive axles in the unloaded configuration (trailer carried on top of vehicle for return trip). Before the start of the test, the vehicles were inspected, serviced, and aligned to factory specifications. Previous road testing confirmed the performance equivalency of both trucks. The tandem torsion bar suspension of the two trucks was adjusted to provide equivalent ride height and, hence, equivalent nominal spring rates.

FIGURE 3 Heavy vehicle operating on test course.

Definition of Failure Modes

For this test, four failure modes were defined, and equal levels of these failure modes were reached before road maintenance was performed. Road maintenance consisted of blading using four to six passes with a water truck and grader, followed by limited wheel compaction with the water truck. Table 1 shows the four failure modes defined for this test and the resulting modes of failure for the four test sections.

NATC's dynamic force measurement vehicle (Figure 4) was used to perform longitudinal profiles of the road roughness (5). An initial "trigger" level was set by having maintenance personnel view and drive the test track. A spectral analysis of the washboard energy was then used to determine when subsequent washboarding had reached the "trigger" level for road maintenance, thus ensuring that all test sections were maintained at an equal level of washboard amplitude.

Summary of Test Results

Blading Ratios

The primary objective was to determine blading maintenance ratios between light-vehicle traffic and logging-truck traffic with both high- and low-pressure tires. The units in Tables 2 and 3 are the number of vehicle laps for an equivalent amount of road damage given the four failure modes. The lap totals are the averages from the individual failure data shown in Table 2. Given the 50-50 split between loaded and unloaded logging-truck traffic, a 1,000-pass reference in Table 2 is the same as 500 round-trips to the sawmill. In Table 2, a ratio greater than 1 means that the logging truck does more damage, whereas a ratio less than 1 means that the light vehicle does more damage. For example, a value of 5.0 means that five passes of a light vehicle are equal in road damage to one pass of a logging truck.

Before the start of this test, estimated blading maintenance ratios between light and heavy vehicles ranged from 3:1 to 96:1 for dry-weather operation depending on the method of calculation and study cited. The data from this test indicate that the blading maintenance ratio ranges between 10:1 to 0.5:1 and varies significantly by grade and alignment. Figure 5 shows the maintenance trends determined from this test. It is important to note that the plane drawn to show the relationship between the blading maintenance ratio and percentage

TABLE 1 Modes of Failure for NATC Test Sections

Failure Mode #1	—	Washboard
Failure Mode #2	—	Four-inch rut depth with limited washboard (from the original ground-line)
Failure Mode #3	—	Material push to the outside of the curve, which tended to super-elevate the curve
Failure Mode #4	—	Combination of potholes/washboard and limited rut depth (worn rut)

Test Section	Light Vehicles	High Pressure Logging Truck	Low Pressure Logging Truck
8% Grade	Washboard	Washboard	Washboard
Curve (ruts)	Ruts	Ruts	Ruts
Curve (out track)	Washboard	Material Push	Material Push
3% Grade	Pothole/Washboard	Pothole/Washboard	Washboard
Natural	Ruts	Ruts	Ruts

grade was drawn from the limited number of data values determined from the test parameters (aggregate gradation, surface thickness, etc.).

Moisture content was approximately 5 percent for the duration of the test, and the subbase was not considered to be a test variable. Whereas a loaded vehicle potentially does more damage to an aggregate road in wet conditions (i.e., ruts, depressions, subgrade damage) (6) the unloaded vehicle—whether a light vehicle or logging truck—does more damage to a road with grade in dry-weather conditions. Therefore, in Figure 5, the empty vehicle was referenced because of the greater potential for washboard damage. In dry weather, the road manager should be aware that the mix of unloaded traffic will have the most damaging effect.

In Table 2, the loaded logging trucks were responsible for the majority of the rut depth and material push in the 27.5-m (90-ft) radius curves, and the higher ratio (9.5:1) reflects this result.

The trend showed that the empty light vehicles ascending the 8 percent grade produced more road deterioration than the empty logging truck according to the results in Table 2. Likewise, the empty light vehicle descending the 3 percent grade resulted in less damage than the logging trucks. The level of washboard damage is a direct function of the wheel torque required to climb the grade and the resulting high wheel slip and tractive hop. In addition, loose (unbound) and unconsolidated material reduces tractive efficiency and increases the energy investment in shear displacement at the tire-ground interface. Washboarding is initiated by this shear displacement. Excessive shear displacement results in cyclical oscillation of the tire and suspension, which further develops the washboard pattern. If traction demand remains constant (i.e., climbing the 8 percent grade), the cyclical change in vertical load (loading and unloading) produces the tractive hop and high wheel slip. Because slip is a function of vertical load, the lighter the axle load under torque, the greater the potential for the mechanisms of washboard formation to start and amplify (7).

In contrast, the unloaded vehicles descending the 3 percent grade required very little torque; therefore, the phenomenon of high wheel slip and tractive hop was

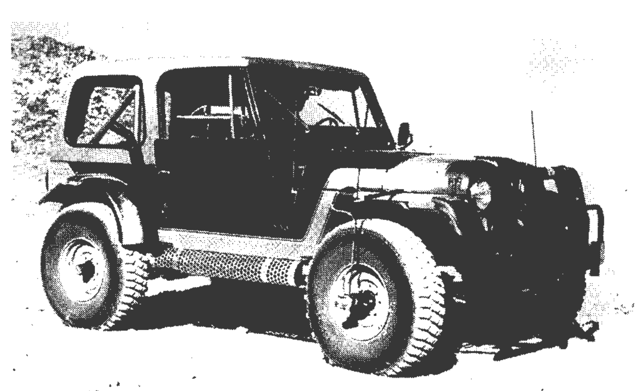


FIGURE 4 Dynamic force measurement vehicle used to determine when maintenance should occur.

TABLE 2 Summary of Blading Maintenance Ratios

INITIAL FAILURE

Test Section	Light Vehicle / High pressure logging truck	Light Vehicle / Low pressure logging truck
8% Grade (favorable)	1262 / 1666 = 0.8	1262 / 3588 = 0.4
3% Grade (adverse)	23978 / 4170 = 5.8	23978 / 8420 = 2.8
Curve (in wheel tracks)	8827 / 921 = 9.6	8827 / 930 = 9.5
Curve (out tracking)	13368 / 3117 = 4.3	13368 / 3663 = 3.6
Native (3% adverse)	(23978) / (4958) = 4.8	(23978) / (8643) = 2.8

SUBSEQUENT FAILURES

Test Section	Light Vehicle / High pressure logging truck	Light Vehicle / Low pressure logging truck
8% Grade (favorable)	561 / 1218 = 0.5	561 / 2746 = 0.2

NOTE: *Lap totals in parentheses represent the passes completed to date but the roughness did not reach the failure criteria.*

never initiated to a level to generate "maintenance level" washboarding. Without cyclical load changes, continuous displacement of road material developed worn ruts rather than washboarding. Worn ruts and limited washboarding were the failure modes on the 3 percent grades (native and aggregate).

For the grade lengths in this test, steady-state torque was required to climb the grade and vehicle momentum was minimized. It should be noted that for grades that are extremely short in length, vehicle momentum is often used to "shotgun" over the shorter grades. Since wheel torque is not applied in a steady-state format given short grade lengths, the results for shorter slopes may differ. When using these ratios and establishing local validation tests, it is recommended that the road network be examined based on 300-ft (91.5-m) sections as the minimum length.

As shown in Tables 2 and 3, the damage ratio relationship decreases as logging truck tire pressures decrease. This is due to the reduced road damage with the use of reduced tire inflation pressures on logging trucks. Table 3 shows the damage ratios between the logging truck with high-pressure tires and the logging truck with low-pressure tires. Table 3 shows that on the straight test sections, the low-pressure logging truck could travel twice the number of passes between maintenance bladings. On the 27.5-m (90-ft), 180-degree curve, the damage was approximately equal.

The trend indicates that the initial "washboard-generated" failure takes longer than subsequent failures. This is potentially because a pattern or reflection is established in the subbase during the initial running that is not removed during subsequent maintenance blading. The pattern accelerates the rate at which the washboard pattern returns. Limited measurements and observations on the high-amplitude washboard cycles showed that the washboarding developed in exactly the same location (longitudinally) with subsequent running after maintenance.

For this test, the maintenance blading restructured the top 7.6 to 10.2 cm (3 to 4 in.) of material given a 15.2-cm (6-in.) aggregate surface. The base material was not disturbed during maintenance blading, and the remaining pattern or reflection of the washboarding helped generate such damage much more quickly with continued running.

TABLE 3 Summary of Blading Maintenance Ratios Between High- and Low-Pressure Logging Trucks

Test Section	High pressure logging truck / Low pressure logging truck
8% Grade (favorable)	1666 / 3588 = 0.5
3% Grade (adverse)	4170 / 8420 = 0.5
Curve (in wheel tracks)	921 / 930 = 1.0
Curve (out tracking)	3117 / 3663 = 0.9
Native (3% adverse)	(4958) / (8643) = 0.6

SUBSEQUENT FAILURES

Test Section	High pressure logging truck / Low pressure logging truck
8% Grade (favorable)	1218 / 2371 = 0.5

NOTE: *Lap totals in parentheses represent the passes completed to date but the roughness did not reach the failure criteria.*

For dry-weather road conditions, the unloaded vehicle did a majority of the washboard-generated damage, especially on the 8 percent grade. This was true for all lanes. Observations on the 8 percent grade showed that the unloaded light vehicles could cause "maintenance-level" washboarding in as few as 74 passes (observed on the sixth and later unofficial failures). The high-pressure logging truck could cause maintenance-level washboarding in as few as 152 unloaded passes (observed on the fifth unofficial failure). This trend indicates that as the road ages from operation in dry-weather conditions, the maintenance cycles will become more frequent. It should be noted that no new aggregate was placed on the test sections during testing (even after the required number of failures); therefore, this trend was noted given the increasingly frequent blading cycles required on the 8 percent grade.

Although no attempt has been made to quantify the ratios given for exclusively unloaded (empty) traffic, the laps to failure would have certainly been significantly fewer for all vehicle types. The loaded vehicles had a tendency to smooth or iron out the washboarding. There is a point, however (especially on the 8 percent grade), where the washboard amplitude reaches a level that the loaded vehicle can no longer smooth out, and washboard failure occurs within the next series of vehicle passes regardless of vehicle load.

On the 8 percent grade, the unloaded vehicles ascended the grade. On the two 3 percent grades, the unloaded vehicles descended the grade. Given that unloaded vehicle traffic ascending a grade caused the majority of the washboarding, the 8 percent grade was at a disadvantage and the 3 percent grades had the advantage from the standpoint of washboard generation. However, for grade operation, the data indicate that there would be a crossover grade where the light vehicle did less damage than the heavy vehicle for an adverse grade, as shown in Figure 5.

For future testing, of limited test time is available, it is recommended that only empty vehicles be used since they tend to cause a majority of the washboarding in dry-weather conditions. Additionally, it is recommended that the unloaded vehicles always ascend the grade.

In a previous Forest Service test conducted by NATC (*8*), the driving scenario required that the logging trucks accelerate on the gravel after coming off the asphalt.

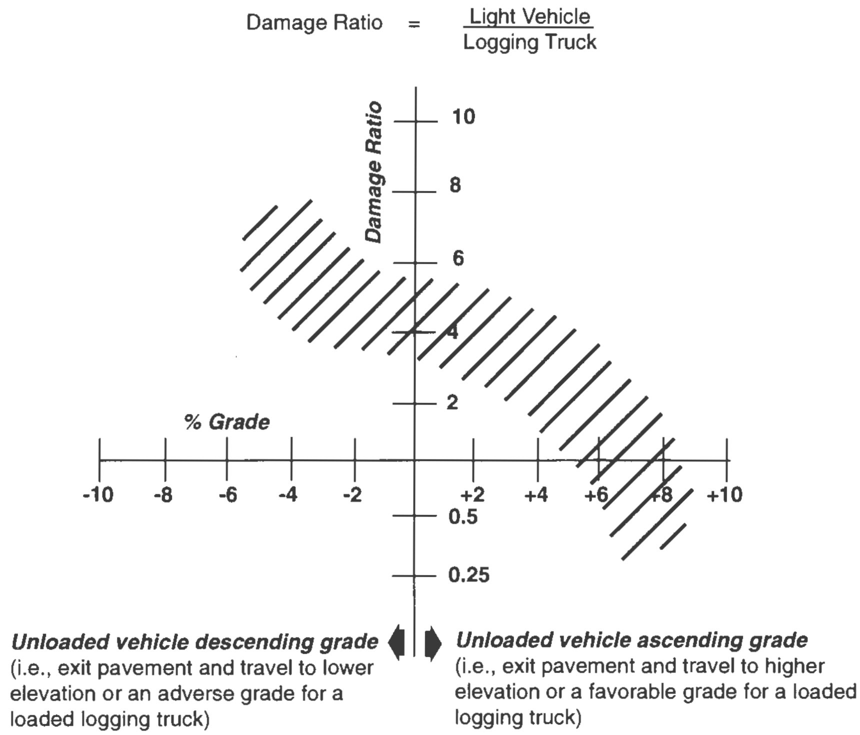


FIGURE 5 Graphical display of maintenance trends for dry-weather vehicle comparisons.

This acceleration generated washboarding quickly because it involved the same energy as climbing a grade. For road geometry that has frequent stops or areas of significant acceleration and deceleration, it is recommended that the ratios measured for the 8 percent grade be used.

It is also important to note that for this test, the vehicle held a constant speed through the curves, and the line of sight around the curves was unrestricted. Sight limitations on curves often dictate that a driver slow down entering the curve and then accelerate exiting it. This driving technique on a blind curve requires wheel torque changes that may reduce the lap totals shown in Table 1.

Finally, it is important to note that the decision to perform maintenance on a road network under actual road conditions may be driven by the highest maintenance area. For example, if the entire road network had grades of less than 3 percent, the ratio would be on the order of 5.8:1 and could involve a high volume of vehicle traffic. Likewise, if the road network had a high percentage of 8 percent grades, the ratio would be on the order of 0.8:1 and a high level of road maintenance would be required to control the roughness on the grades. These relationships need to be tailored based on the local road networks.

Material Loss Ratios

A second objective was to determine volume loss ratios between light-vehicle traffic and logging-truck traffic. Volume loss was a combination of many variables, such as dust, material separation (loss of fines), stone throw, material push to the outside of the curve, unrecoverable material lost due to blading, and so forth. Tables 4 and 5 show the volume loss ratios in cubic feet of material displaced as a result of the laps in Table 2. A negative value means that an overall material loss was measured. A positive value means that an overall material gain was measured. A material gain occurred when compacted material was displaced due to vehicle traffic and the uncompacted material consumed more volume in its berm form. This occurred predominantly in the curved test sections. For the straight test sections, the material loss or gain was that measured over the center 100 ft of the test section. For the curved sections, the material loss or gain was that measured over the entire 180-degree curve or 283 ft. For this analysis, transverse profiles measured from the 24 temporary benchmarks (TBMs) were averaged into a single plot to represent the entire test section.

Note that in this volume loss ratio analysis, the results are inverted from the maintenance blading ratios.

TABLE 4 Summary of Volume Loss Ratios

Test Section	Light Vehicle / High pressure logging truck	Light Vehicle / Low pressure logging truck
Ratio Units are Volume Loss (M^3) per Pass X 1000		
8% Grade (favorable)	$\frac{0.493}{0.719} = 0.7$	$\frac{0.493}{0.716} = 0.7$
3% Grade (adverse)	$\frac{0.079}{0.261} = 0.3$	$\frac{0.079}{0.346} = 0.2$
Curve (in wheel tracks and outtracking)	$\frac{0.201}{1.731} = 0.1$	$\frac{0.201}{2.450} = 0.08$
Native (3% adverse)*	$\frac{0.045}{0.235} = 0.2$	$\frac{0.045}{0.119} = 0.4$

** Measured at 4958, 5523 and 15,001 passes for the high pressure, low pressure and light vehicle lanes, respectively.*

For the volume loss ratios, a value less than 1 means that the logging-truck traffic has a higher amount of volume loss per pass. For the straight test sections, the trend is similar to the maintenance blading ratios shown in Table 2 and ranged from 0.2 to 0.7, depending on the grade and alignment (indicating that a logging truck produces 1.5 to 5 times more volume loss per pass than a light vehicle). Similar to the maintenance blading ratios, the pickup and logging-truck ratios are closer to unity when the unloaded vehicles ascend the 8 percent grade.

In Table 4, the loaded logging trucks were responsible for the majority of the rut depth and material push road damage in the 27.5-m (90-ft) radius curves, which resulted in higher volume lost per pass. As a result, the ratios are higher for the curves than those shown in Table 2.

The material loss data from the curved test sections were inconclusive on an individual failure basis. The material loss data for the test scenario where the trucks stayed in their wheel paths showed a net gain in material for the first failure and a net loss for the second failure. It is hypothesized that the material gain came from removal of compacted material in the curves (due to the ruts or material push) and creation of berms of uncompacted material by the trucks. The uncompacted material had more volume, thus the overall gain of material. This volume gain trend was also measured for the test scenario where the trucks out-tracked with each vehicle pass. The trend showed that the logging trucks pushed more material in the curves than the light vehicles. The trend also showed that the low-pressure logging truck pushed more material than the high-pressure logging truck when the vehicle out-tracked with each vehicle pass. These volume loss ratios were 0.16 and 0.05 for the ratio between the light vehicle and the high- and low-pressure logging trucks, respectively. This means that the high-pressure logging truck has 6.3 times more volume loss than the light vehicles in the curves, and the low-pressure logging truck had 20 times more volume loss. This increased material push with the low-pressure logging truck was attributed to the longer footprint of the low-pressure tires.

TABLE 5 Summary of Volume Loss Ratios Between High- and Low-Pressure Logging Trucks

Test Section	High pressure logging truck / Low pressure logging truck
8% Grade (favorable)	$\frac{0.719}{0.716} = 1.0$
3% Grade (adverse)	$\frac{0.261}{0.346} = 0.8$
Curve (in wheel tracks and outtracking)	$\frac{1.731}{2.450} = 0.7$
Native (3% adverse)	$\frac{(0.235)}{(0.119)} = 2.0$

NOTE: *Lap totals in parentheses represent the passes completed to date but the roughness did not reach the failure criteria.*

After two material push (out-tracking) road failures, the low-pressure logging truck lane was significantly superelevated, and little material remained on the inside of the curve (the subgrade material was exposed). The curve for the low-pressure logging truck would have required material replacement if additional running was to be conducted. Given 126.7 m^3 (4,525 ft^3) of material placed in each curve test section at the time of construction, the total volume lost for the low-pressure logging truck equaled 25 percent of the total volume.

The curves were the only test sections that experienced unrecoverable material loss due to blading. The superelevated material was pushed beyond the TBMs established for the transverse profile measurements. It should be noted that the straight test sections had 5 ft of additional road surface beyond the outside of the lanes; therefore, material lost in the ditch due to blading was not a factor in this test.

Rut-Depth Ratios

A third objective was to determine rut-depth ratios between light vehicle traffic and high- and low-pressure logging truck traffic. Tables 6 and 7 show the rut-depth ratios in average maximum rut depth in inches as a result of the laps in Table 2. For this analysis, 24 transverse profiles were averaged into a single plot to represent the entire test section. The rut-depth value reported was the maximum difference between the repaired profile before vehicle traffic and the damaged profile after vehicle traffic. In the case of the material push, where no dominant ruts developed, the rut depth reported was the maximum difference between the repaired and damaged profiles.

Note that in this rut-depth ratio analysis, the results are also inverted from the maintenance blading ratios. A value less than one means that the logging truck traffic has a greater rut depth per pass. For the straight test sections, the trend is similar to the maintenance blading ratios in Figure 5 and ranged from 0.2 to 0.8, depending on grade and alignment (indicating a logging truck produces 1.3 to 5 times more rut depth per pass than a light vehicle). Similar to the maintenance blading ratios, the light-vehicle and logging-truck ratios are closer to unity when the unloaded vehicles ascend the 8 percent grade.

Conclusions

This test indicated that many variables come into play regarding determination of road damage ratios by vehicle type. For the dry-weather conditions encountered, the light vehicle ascending a grade causes increasing damage as the road steepens. Additionally, for heavy-vehicle operations, the use of reduced tire pressure almost always results in a decrease in road damage.

TABLE 6 Summary of Rut Depth Ratios

Test Section	Light Vehicle / High pressure logging truck	Light Vehicle / Low pressure logging truck
Ratio units are maximum average rut depth (mm) per pass X 1000		
8% Grade (favorable)	$\frac{38.1}{45.7} = 0.8$	$\frac{38.10}{20.32} = 1.9$
3% Grade (adverse)	$\frac{2.54}{11.68} = 0.2$	$\frac{2.54}{5.84} = 0.4$
Curve (in wheel tracks)	$\frac{7.62}{99.06} = 0.08$	$\frac{7.62}{93.98} = 0.08$
Curve (out tracking)	$\frac{4.06}{33.02} = 0.12$	$\frac{4.06}{30.48} = 0.13$
Native (3% adverse)*	$\frac{(2.29)}{(8.64)} = 0.3$	$\frac{(2.29)}{(3.56)} = 0.6$

** Measured at 4958, 5523 and 15,001 passes for the high pressure, low pressure and Light vehicle la ıes, respectively.*

NOTE: *Lap totals in parentheses represent the passes completed to date but the roughness did not reach the failure criteria.*

TABLE 7 Summary of Rut Depth Ratios Between High- and Low-Pressure Logging Trucks

Test Section	High pressure logging truck / Low pressure logging truck
8% Grade (favorable)	45.7 / 20.32 = 2.3
3% Grade (adverse)	11.68 / 5.84 = 2.0
Curve (in wheel tracks)	99.06 / 93.98 = 1.1
Curve (out tracking)	33.02 / 30.48 = 1.1
Native (3% adverse)	(8.64) / (3.56) = 2.4

NOTE: *Lap totals in parentheses represent the passes completed to date but the roughness did not reach the failure criteria.*

Four modes of road deterioration were defined for dry-weather operation: washboarding, worn ruts, material push, and potholes.

Before the start of this test, estimated blading maintenance ratios between light and heavy vehicles ranged from 3:1 to 96:1 for dry-weather operation, depending on the method of calculation. The data from this test indicate that the blading maintenance ratio ranges between 10:1 to 0.5:1 and varies by grade and alignment.

On the 8 percent grade test sections, the empty vehicles ascending the grade caused the majority of the washboard damage, and the light vehicles did more damage than the logging trucks (0.5:1 ratio).

On the curved test sections, the loaded logging trucks did the majority of the damage, resulting in light- to heavy-vehicle maintenance ratios between 4:1 to 10:1, depending on the width of the road.

On tangent test sections, the blading maintenance ratio showed that a low-pressure logging truck would require half the road maintenance of a high-pressure logging truck. On curved test sections, the blading maintenance ratio was approximately equal. The same was true of the ratios for average rut depth per pass.

In dry-weather conditions, the primary trigger for road maintenance was washboarding. The data showed that the initial maintenance cycle required a higher number of passes to achieve the same level of washboarding than did the subsequent failures. The subsequent washboard-generated maintenance cycles occurred much more frequently due to the washboard pattern remaining in the road material below the depth of blading. Under continuous dry-weather conditions, the laps between subsequent maintenance cycles decreased.

REFERENCES

1. Gibby, K., R. Kitamura, and H. Zhao. Evaluation of Truck Impacts on Pavement Maintenance Costs. In *Transportation Research Record 1262*, TRB, National Research Council, Washington, D.C., 1989, pp. 48–56.
2. Paterson, W. *Road Deterioration and Maintenance Effects: Models for Planning and Management.* Highway Design and Maintenance Standards Series, World Bank. Johns Hopkins Press, Baltimore, Md., 19 .
3. Paige-Green, P., and A. Visser. Comparison of the Impact of Various Unpaved Road Performance Models and Management Decisions. In *Transportation Research Record 1291*, TRB, National Research Council, Washington, D.C., 1990.
4. Ashmore, C. *Draft Report: Road Maintenance Commensurate Share Test.* Nevada Automotive Test Center, Carson City, 1994.
5. ARE, Inc. *Surfacing Thickness Guide.* Forest Service, U.S. Department of Agriculture, 1990.
6. Hodges, H. C., et al. *Nevada Automotive Test Center Final Report—Central Tire Inflation.* Report 53-9JA9-6-SD647. Forest Service Technology and Development Center, U.S. Department of Agriculture, San Dimas, Calif., 1987.
7. Ashmore, S. C., and H. C. Hodges, Jr. *Dynamic Force Measurement Vehicle (DFMV) and Its Application to Measuring and Monitoring Road Roughness, Vehicle, Tire, Pavement Inferface.* In ASTM STP 1164 (J. J. Henry and J. C. Wambold, eds.), American Society for Testing and Materials, Philadelphia, Pa., 1992, pp. 69–96.
8. Della Moretta, L. D., and H. C. Hodges. Off-Highway Tire/Road Damage and Healing Mechanism. Presented at Technical Seminar on Off the Road Uses of Tires, Akron, Ohio, 1982.

Open-Graded Emulsion Mixtures: 25 Years of Experience

R.G. Hicks, *Oregon State University*
Emory S. Richardson, *Federal Highway Administration*
I.J. Huddleston, *Asphalt Paving Association of Oregon*
Newton C. Jackson, *Lacey, Washington*

Open-graded emulsion mixtures (OGEM) have been used extensively as road surfaces for low-volume roads in the Pacific Northwest since 1966. They consist of open-graded aggregate (20 × 6 mm) and 5 to 7 percent of a cationic mixing graded emulsion (CMS-2 or CMS-2h). This paper presents the results of several agency experiences indicating that OGEM usually performs as well as or better than conventional dense-graded asphalt concrete in most applications. It also includes mixture properties (modulus, changes in gradation with time, etc.) to illustrate that conventional test procedures are not appropriate for evaluating this product and layer equivalencies that have evolved during this period, as well as guidelines for use of OGEM. Well over 16 000 km (10,000 mi) of roads has been surfaced with OGEM during this period.

Open-graded emulsion mixtures (OGEM) have been used extensively in the Pacific Northwest since 1966 (*1*). Aggregates used for OGEM generally pass the 25-mm (1-in.) sieve, with 0 to 10 percent passing the 2-mm (No. 10) and 0 to 2 percent passing the 0.074-mm (No. 200) sieves. The emulsified asphalts normally used have been CMS-2 or CMS-2h. Well over 16 000 km (10,000 mi) of two-lane roads has been constructed using an open-graded aggregate with CMS-2 or CMS-2h emulsions. The first performance survey of OGEM was conducted in 1976 and evaluated several projects throughout Oregon and Washington (*2*). Each project was rated for ride quality using a passenger car driven at normal speeds. Close-up inspections were made to identify types of distress and to arrive at an overall evaluation for each project. In general, all pavements surveyed were in good condition (an overall rating of 7.9 out of 10). The types of distress observed included rutting (less than 9.5 mm), alligator cracking, raveling of the surface treatment, and poor ride quality. The poor ride was generally attributed to roughness built in during construction. Construction information, traffic data, and materials information were obtained for each project. Cores were taken from each project to determine the resilient modulus, aggregate gradation, and residual asphalt content and properties. On the basis of the 1976 survey, it was concluded that several factors other than traffic affect performance, including environment, quality control during construction, subgrade and base type, and drainage.

Similar surveys were completed in 1981 (*3*) and 1986 (*4*). Several significant findings resulted from these surveys:

• There was little change in performance for most roads, although distress in the form of thermal cracking began to appear on projects in the colder regions of Oregon and Washington.

• The ride quality of open-graded emulsified asphalt pavements was as smooth as that on hot-mix pavements. However, if proper construction practices are not followed, the mix can prematurely stiffen, producing a rough surface during laydown.

• Mix modulus increased with pavement age; calculated asphalt penetrations decreased with age. However, the data were scattered due to variations in core density, void content, asphalt content, climate, and surface seal conditions.

• Cores with high asphalt penetrations gave low moduli or stiffnesses. The converse was not always true. Several cores with low moduli also had low calculated asphalt penetrations.

• Projects with high calculated asphalt penetrations (greater than 20 dmm) had a lower incidence of distress than those with penetrations of 20 dmm or less. The incidences of distress were about equal between projects having high and low mix modulus.

• Structural layer coefficients calculated from this survey indicate open-graded emulsified asphalt pavements have a thickness equivalency of 0.9 to 1.0 when compared to hot-laid asphalt pavements.

Objectives

The purposes of this study were to

• Review and update the performance of projects constructed by three agencies in the Pacific Northwest;

• Present typical mixture properties for these mixtures;

• Compare the performance of OGEM with conventional hot mix asphalt concrete, including the development of layer equivalency factors; and

• Present the general guidelines for use of OGEM adopted by each agency in the Northwest.

Performance of OGEM

Historical Perspective

Three performance surveys have previously been conducted to evaluate the relative performance of OGEM. In July 1986, 19 projects were surveyed, including the 12 projects surveyed in August 1976 and September 1981. The survey technique was identical to that employed in 1981. Cores were taken to determine the density, modulus, aggregate gradation, and residual asphalt content and properties.

Construction Information

Table 1 summarizes the construction data for each of the 19 projects surveyed in 1986. As indicated, projects were constructed by a number of agencies and ranged from less than 1 to 20 years in age. A sample of 236 km (147.5 mi) of open-graded emulsified asphalt pavements was surveyed. In all cases, the open-graded mix was placed as the wearing surface with thicknesses ranging from 5 to 23 cm (2 to 9 in.). Chip seals were added later to improve the resistance to raveling and to provide waterproofing. The projects surveyed included mixes used in new construction as well as for overlays.

Traffic Data

Traffic data for each of the 19 projects were collected from the responsible agency. These data were used to calculate the number of 80-kN (18,000-lb) equivalent single-axle loads (ESALs). The procedure used for calculating the ESALs is provided by Hicks et al. (*4*). Most of the roads surveyed had ESALs of less than 1 million, with maximum ESALs of about 4 million.

Performance Data and Maintenance History

All projects were rated for overall evaluation, ride quality, and type and severity of distress. The overall evaluation for most of the projects was "good" to "very good" with only a few of the projects rated as fair. Though most of the projects gradually dropped in rating with time, a chip seal generally restored the overall evaluation to "very good" (*4*).

Ride quality was rated as "fair" to "very good" on most projects. The older projects received "fair" ratings, while the newer projects received "very good" ratings, giving smooth quiet rides. Most of the sections receiving "fair" ratings were rough shortly after construction due to laydown difficulties; they have not changed much since. Observations of particular interest include the following:

• The most common type of distress noted was minor pitting or raveling of the surface. This is normally corrected with the use of a chip seal.

• Most of the remaining projects showed little change in appearance from that observed in 1981.

• Performance is generally better than hot mix in the eastern parts of Oregon and Washington (*2–4*). Specifi-

TABLE 1 Construction Information on OGEM Surveyed in 1986

Project	Responsible Agency	Date Constructed	Project Length (km)	Mix Thickness (cm)	Emulsion Type
Lewis River	Gifford Pinchot National Forest	1970	25.1	17.5 to 23	CMS-2h
Hermiston-Meadow Valley Interchange	ODOT Region 5	1969	7.2	7.5	CMS-2h
Blue Mountain-Elgin*	ODOT Region 5	1983	26.7	10	CMS-2
Palmer Junction*	FHWA (WDFD)	1985	13.5	10	CMS-2h
Elgin-Minam	ODOT Region 5	1981	14.5	10	CMS-2
Vance Creek Rest Area-Cottonwood Creek*	ODOT Region 5	1984	31.2	6	CMS-2
Logan Valley	Malheur National Forest	1971	17.9	10	CMS-2h
Dayville-Flat Creek*	ODOT Region 5	1984	9.8	5	CMS-2
Painted Hills-Mitchell*	ODOT Region 4	1981	10.5	10	CMS-2h
FHWA-Ochoco Summit*	FHWA (WDFD)	1981	7.4	10	CMS-2h
Hogback Summit	ODOT Region 4	1976	12.2	7.5	CMS-2s
Diamond Lake Bypass	FHWA (WDFD)	1977	10.5	7.5	CMS-2h
Indian Caves-Medicine Creek	Umpqua National Forest	1971	10.9	20	CMS-2h
Umpqua Community College	Douglas County	1967	1.9	7.5	CMS-2h
Cow Creek Canyon	Douglas County	1973	11.9	6	CMS-2
Clarks Branch Road	Douglas County	1976	3.7	5	CMS-2h
Smith River	Douglas County	1967	5.3	6	CMS-2h
Mapleton	FHWA (WDFD)	1978	7.4	19	CMS-2h
Decker Road*	Benton County	1979	9.7	10	CMS-2
Total Kilometers			237.4		

*New Projects
1 inch = 2.54 cm
1 mile = 1.61 km

cally, OGEM provides better resistance to fatigue and thermal cracking and to water-related distress (e.g., stripping).

Oregon DOT Projects

The Oregon Department of Transportation (DOT) constructed the first OGEM pavements in the late 1960s. Since that time, more than 650 km (404 mi) has been laid, accounting for more than 1.8 million tons of mix (this does not include work by counties). Although the performance of the early jobs was very good, the use of OGEM was sporadic until the mid-1980s. This was primarily due to a belief that the material was structurally inferior to dense hot-mix asphalt (DHMA). It was also caused by some jobs' exhibiting poor ride quality.

In the early 1980s, the Oregon DOT began to use OGEM at a much higher rate. This was due to the past excellent performance, low cost, environmental benefits, and changed structural design criteria. Oregon DOT engineers found that the early OGEM jobs had experienced less fatigue and thermal cracking and less moisture damage than their DHMA counterparts. The jobs completed in the late 1960s performed well for more than 25 years before they were finally overlaid. All other OGEM projects constructed since the early 1970s are still in service and are in fair to good condition.

The early design criteria used by the Oregon DOT assigned layer coefficients to OGEM that were 10 to 15 percent less than those for DHMA. However, around 1984, the design procedure was modified to allow equal structural layer coefficients for these materials in all new works. For overlay designs, the Oregon DOT also began to allow a significantly higher tolerable deflection for OGEM overlays than for DHMA overlays, in recognition of the excellent fatigue and reflective crack resistance experienced on early projects. The increased structural credit in conjunction with low initial construction cost has resulted in more jobs being constructed with OGEM. Currently, the Oregon DOT is placing more than 300,000 tons (270 000 tonnes) of OGEM per year. These projects are primarily conducted on low- to medium-volume highways [with average daily traffic (ADT) of less than 5,000] in the eastern half of the state where the environmental conditions are most extreme. The typical low winter temperatures in this portion of the state range from −1°C to −3°C and can be as low as −4°C locally. OGEM resists thermal cracking in this environment very well.

Construction practices have also improved for OGEM. Although some jobs were constructed with poor ride quality, in most cases the ride quality on OGEM is comparable to DHMA. The Oregon DOT is in the process of implementing a new smoothness specification for asphalt concrete pavements, which uses a California-type profilograph to measure smoothness. This specification will apply equally to OGEM and DHMA. Both pavement types must be constructed with a profile index of 11 cm/cm (7 in./in.) per mile or less to receive full pay.

Washington DOT Experience

The first open-graded emulsion pavement constructed in Washington was by change order on a 1975 project near the town of Moxee in the central part of the state. Table 2 summarizes the Washington DOT projects using this material through 1991. Several counties within the state have also used this material for many years, but their projects have been less well documented and are not included in the listing.

As seen in Table 2, the Washington DOT had a fairly aggressive start using this material in 1976 when over 50,000 tones (45 000 tonnes) were placed in four contracts. Unfortunately, a recurring inability to obtain a smooth ride on all of the projects somewhat discouraged the use of the material. Several years elapsed before the state began using OGEM again. By the mid-1980s, the very good performance of this material in resisting rutting, fatigue cracking, and thermal cracking renewed interest in its use, and the number of contracts using OGEM increased markedly after 1984. With more attention given to basic construction details on the part of the state, contractor, and emulsion suppliers, the ride quality of OGEM has improved on most of the subsequent projects.

A few OGEM projects had poorer performances than the rest of the OGEM projects. These projects were located on routes with higher truck traffic volumes that seemed to have been constructed with somewhat lower emulsion contents. Project personnel have subsequently been encouraged to keep the emulsion rates up just short of having runoff problems. The districts are also now encouraged to fog-seal all OGEM pavements every 5 years similar to hot mixed open-graded friction courses.

WSPMS is a project-specific pavement management system that uses annual pavement condition surveys, collected on the entire state highway system grouped by project length segments, to predict pavement performance and project rehabilitation timing. The pavement condition is represented by a pavement structural index that uses the severity and extent of fatigue cracking, transverse cracking, and patching in a weighting system that produces a scale of 0 to 100. The scale is calibrated for flexible pavements so that a value of 100 indicates no visible distress. A value of 50 indicates the classical level of 10 percent fatigue cracking, with additional small amounts of transverse cracking and patching similar to a present serviceability index (PSI) of 3.0. Values approaching 0 represent an accumulation of severe levels of all distress similar to a PSI of between 2.0 and 1.5. In Washington State, the construction program is based on resurfacing or rehabilitating most pavements when they reach the level of 50. This system identifies the service life of any pavement section as the number of years from last construction or resurfacing until it reaches the structural condition index (SCI) of 50. Four categories of pavement condition are used by the state DOT to describe pavement condition. These descriptions are "good," "fair," "poor," and "very poor," which describe pavements with a condition rating from 100 to 75, 75 to 50, 50 to 25, and 25 to 0, respectively.

The average service life to an SCI of 50 for asphalt concrete pavement (ACP) in District 5, where most of the emulsion projects are located, is 11.2 years. An evaluation of the WSPMS data for the first eight OGEM projects constructed between 1975 and 1982 indicates that the service life to a rating of 50 ranged from 12 years to 21 years, with an average life of 14.1 years. Thus, the average service life of the first eight OGEM projects constructed in Washington was 25 percent longer than the average service life for ACP in the same area. Since the state's pavement structural index primarily indicates fatigue performance, rut depth and ride values were also checked for each of the eight projects. The maximum rut depth measured 6 mm (0.25 in.) with little or no rut indicated on most projects. There was also no discernible change in the ride values measured since construction on any of the projects at this time.

The first several OGEM projects were designed using normal design procedures, except that a thickness equivalency of 1.15 compared to 1.00 for ACP was used. In subsequent projects, the thickness equivalency

TABLE 2 OGEM Projects in State of Washington

Cont. No.	Dist. No.	SR No.	Project Name	Section Length, mi	Tonnage	Year Paved	1993 Condition
9689	5	24	Moxee Vicinity	1.37	8,665	1975	Fair
9941	5	97	Klickitat County Line to Dry Creek Bridge	6.37	13,800	1976	Chip sealed in 1983
0180	5	97	Dry Creek to Oak Springs Road	0.99	4,720	1976	Overlaid in 1991
0232*	5	12	Patit Creek to Willow Creek Hill	5.43	17,430	1976	Chip sealed in 1991
0240*	6	272	Colfax to Idaho State Line	5.40	16,500	1976	Chip sealed in 1993
0425**	5	970	East Cle Elum I/C to Teanaway River	5.66	22,500	1978	Fair
1982	5	24	SR-241 to BPA X-ing	3.64	3,420	1981	Good
2262*	6	127	Dodge to Meadow Creek Summit	6.02	18,577	1982	Good
2716	5	22	Yakima County Line to SR-82	6.32	26,598	1984	Good
2736	3	121	SR-12 in Rochester to SR-5	3.40	5,230	1985	Fair
2875	5	395	SR-260 to Adams County Line	6.42	28,000	1985	Fair
3885	5	221	Sellars Road to County Well Road	3.42	10,130	1985	Poor
2927	5	17	Adams County Line to M.P. 12	9.40	37,886	1985	Poor
2980	4	14	Klickitat River to Horse Thief Canyon	10.44	19,397	1987	Good
3080	5	12	Airport East I/C to Weight Station	1.03	4,990	1987	Good
3232	5	221	Prosser Hill	2.83	8,822	1987	Good
3263	2	17	West Foster Creek Bridge to East Foster Creek	5.58	8,920	1987	Fair
3280	5	970	Teanaway River to SR-7/Virden	4.66	11,160	1987	Overlaid in 1991
3284	5	12	Vansycle Canyon to Nine Mile Canyon	2.86	5,247	1987	Fair (with winter snow plow damage)
3289	5	12	Fairview road to M.P. 416.8	3.30	2,700	1987	Good
3269	4	12	Slide Br. to Yakima County Line	3.30	14,920	1988	Good
3357	5	410	Bumping River Road to Lower Nile Road	4.09	6,204	1988	Fair
3427	5	12	SR-261 to Archer Road	4.30	12,236	1988	Good
3419	5	12	M.P. 319.34 to Lower Dry Creek Road	5.41	11,850	1988	Fair
3562	5	124	Ash Road to M.P. 17.50	6.32	18,445	1989	Good
3808	5	12	Mud Creek Road to Road 1275	2.91	7,960	1990	Good
3927	5	12	SR 730 to Van Sycle Canyon	3.79	8,300	1991	Good
3915	5	97	Vic. Bridge 97/102 to M.P. 38.1	2.67	9,546	1991	Good
3813	5	97	M.P. 38,09 to M.P. 41.27	3.18	8,200	1991	Good
3884	5	221	Lenzie Road to Sellards Road	7.18	25,170	1991	Good
3875	6	28	Lamme to Harrington	14.6	43,800	1991	Good
			Total	152.29	441,323		

*Was resealed.

**Was overlaid with 12 mm (½ in.) depth friction course at the end of the project to correct roughness.

1 mi = 1.61 km

TABLE 3 FHWA Projects Using OGEM

Project Name	Length (km)	Thickness (cm)	Tonnage	Date Constructed	Present Condition
Lewis River Road N-90	25.1	20-25.4	85,235	1970	Overlaid in 1990
Elk Creek Road	18.3	20	55,240	1971	N/A
Clackamas Highway	2.2	15	4980	1975	Overlaid in 1980
Cascade Lakes Highway	10.5	7.6	16,870	1976	Chip sealed in 1980
Siuslaw Highway	7.4	19	36,650	1976	Fair to Good
Moon Creek	9.0	9-23	22,075	1977	Fair
Nestucca River Road	4.8	variable	N/A	1977	Fair
Ochoco Highway	7.5	18	19,810	1981	Fair
Santiam Highway	12.8	10	19,320	1982	Reconstructed
Palmer Junction	13.4	10	26,500	1985	Fair
Burns-Izee	13.9	10	27,405	1990	Very Good
Burns-Izee	16	4	32,000	1993	Very Good

1 inch = 2.54 cm
1 mi = 1.61 km

was changed to 1.00 to 1.00. Around 1984, the layer equivalency value was changed to 0.85 to 1.00 for ACP because emulsion projects were showing little or no defects. Over the last few years, Washington State has been using a mechanistic-based overlay design method developed by the University of Washington (5). When using this design procedure for OGEM pavements, the shift factor for fatigue damage is adjusted to allow 25 percent more strain than is used with the normal ACP hot mix. This adjustment was based on studies in Sweden that indicated the emulsion mix tolerated much higher strain levels to comparable levels of fatigue (6). The Swedish work seems to be supported by Washington State's performance data for OGEM. This adjustment in the fatigue damage criteria, as used in the mechanistic-based overlay design procedure, is very similar to using a thickness equivalency value of 0.85 to 1.00 in the more standard empirically based pavement design methods.

FHWA Experience

FHWA began constructing OGEM pavements in 1970 and continued through the early 1980s. Construction and early performance problems in the early 1980s caused the agency to reevaluate its use of OGEM. New projects were constructed in the 1990s as shown in Table 3. Following is an annotated summary of the projects constructed to date.

Lewis River Road N-90

Constructed in 1970, this was the first major project using open-graded emulsified asphalt pavement in the Pacific Northwest. The pavement outperformed the calculated design life. Logging trucks in excess of 890 000 kN (200,000 lb) traveled this road. The project was widened and reconstructed in 1990.

Elk Creek Road

Constructed in 1971 this was a timber access road using 20 cm (8 in.) of OGEM directly on a clay subbase. This road is still in place today. Distress due to landslides required local corrective measures.

Clackamas Highway

Constructed in 1975, this project was a test section using three types of emulsified asphalt pavement construction. This was a national experimental evaluation project evaluating open-graded mix using CMS-2h, sand mix using CMS-2s, and a dense mix using CSS-1 emulsion. This project was overlaid with conventional asphalt mix after 5 years of evaluation. The emulsion mixes were used as a bituminous stabilized base at the conclusion of the study.

Cascade Lakes Highway

Constructed in 1976, this was the smoothest pavement constructed so far. The comparison included conventional hot-mix pavements in addition to the OGEM mixes. Due to the altitude and chain wear, a seal coat was placed on the surface about 1980. The road is still performing today.

Siuslaw Highway

Constructed in 1976, this project had a seal placed between lifts that allowed for drainage of the pavement surface to eliminate spray and minimize hydroplaning. This area has rainfall in excess of 250 cm (100 in.) per year. The pavement provided excellent drainage and has performed well. A portion of the structure was placed over floating peat bogs and vertical movement was noticeable. The pavement did not crack but warped to follow the movement of the bog.

Moon Creek

Constructed in 1977, these test sections were used to determine the layer equivalency of open-graded mixes. The sections were constructed with variable thicknesses of pavement. The thin lift sections were calculated to fail. Using traffic data, we planned to calculate structural equivalency. Due to lack of viable traffic data for the calculations, the layer equivalency was not sufficiently accurate to publish in a formal report. The data substantiated that the OGEM could be structurally designed using equal strength coefficients.

Nestucca River Road

In 1977 this road was constructed using marginal marine basalt, which does not perform well using conventional construction methods with dense asphalt systems. Short sections of conventional dense hot mix were placed for comparison to the OGEM. The OGEM performed satisfactorily, while the conventional mix was overlaid within a year. A major construction technique evolved from this project when using marine basalt: marine basalt must be used during the same construction season in which it was crushed. Initially, marine basalts were allowed to be stockpiled from one construction season to the next. This proved unsatisfactory since the gradation was altered by degradation while in the stockpile (e.g., the P200 increased over the winter from 1.5 percent P200 to about 6 percent). Therefore, it was found that encapsulating the aggregate with a thick film of emulsion as soon as practical would minimize the effects from degradation.

Ochoco Highway

Constructed in 1981, this major forest highway is still serviceable today. The road was constructed with some roughness, which was attributed to construction techniques. Though the division engineer of Western Federal Lands Highway Division was not pleased with the rough ride, the pavement performance has proven satisfactory (i.e., it is still performing well in 1994).

Santiam Highway

Constructed in 1982, this highway was totally reconstructed and the OGEM eliminated from the surface. The project required 100 mm (4 in.) of OGEM. The first 50 mm (2 in.) was placed and construction was terminated for winter shutdown. The 50-mm (2-in.) layer was inadequate for the high volume of truck traffic, and numerous sections failed during the winter. Reconstruction took place the following construction season with conventional hot mix 100 mm (4 in.) deep.

Palmer Junction

The original 1985 pavement has been reconstructed. Numerous subgrade problems were not identified in the original construction and had to be dug out. Also, the CMS-2h emulsion was manufactured in such a manner that the emulsion took over a month to break. It is important to have consistency in the manufacturing of the product so the user can anticipate what to expect in cure time.

Burns-Izee

Constructed in 1990, this project was awarded second place by the Oregon Asphalt Paving Association for smoothness and aesthetics. Another Burns-Izee project was constructed in 1993.

Mixture Properties

Cores were taken from most of the projects in each of the three surveys. All cores were tested for density and modulus by Oregon State University and for extraction and measurement of the binder properties by Chevron USA.

Mix Properties

The results of the tests on cores taken over the study period indicated the following:

- Core densities range from about 14.0 to 17.3 N/m^3 (111 to 137 lb/f^3).
- Air voids range from about 13 to 30 percent.
- Extracted asphalt contents range from about 3.0 to 5.8 percent.
- Percentage passing the 0.074 mm (No. 200) ranges from 2 to 5 percent with the percentage passing the 2 mm (No. 8) ranging from 10 to 20 percent.

These results indicate that there is some change in properties over time. Void contents have dropped from 25 to 30 percent to 13 percent and P20 has increased from 0 to 2 percent to 2 to 5 percent.

Modulus Data

All modulus tests were performed using a repeated load diametral test (ASTM D-4123). The results (given in Figure 1) indicate the following:

- In most cases, modulus values tended to increase with pavement age.
- Many of the older projects have modulus values exceeding 68.9 GPa (10^6 psi).
- The 1986 modulus values for selected projects (e.g., Hogback Summit and Elgin–Blue Mountain Summit) were extremely low.
- Figure 2 shows that for most projects sampled in 1976, 1981, and 1986 there is an increase in modulus, with the exception of Hogback summit.

Recovered Asphalt Data

The properties of asphalts recovered from cores obtained in the 1981 and 1986 surveys indicate the following:

- Penetration of the recovered asphalt generally decreases with pavement age (Figure 3).
- For projects sampled in 1981 and 1986, the penetration, in most cases, decreased with time (Figure 4).
- The penetration of some of the older projects is less than 20 dmm.
- The penetration for selected projects (e.g., Clarks Branch Road and Cow Creek Canyon) appears to be extremely high.

Evaluation of Results

Properties Versus Performance

Mix modulus would be expected to increase with time as the pavements age. In general, this occurred but with a significant scatter of results, as shown in Figure 1. The scatter is not too surprising since other variables (e.g., core density, air void content, asphalt content, amount of seal over the open-graded pavement, and the climate conditions) are not equal.

The penetration of the recovered binder decreased with age, as shown in Figure 3. With the exception of results from the Cow Creek Canyon and the Clarks Branch Road projects, the correlation is quite good. Projects over 10 years old have recovered penetrations of 5 to 20 dmm, while projects under 10 years old generally produced recovered penetrations greater than 20 dmm. This suggests the importance of periodic fog seals to rejuvenate the binder.

Recognizing that other variables are not equal, mix modulus versus penetration of the recovered binder are

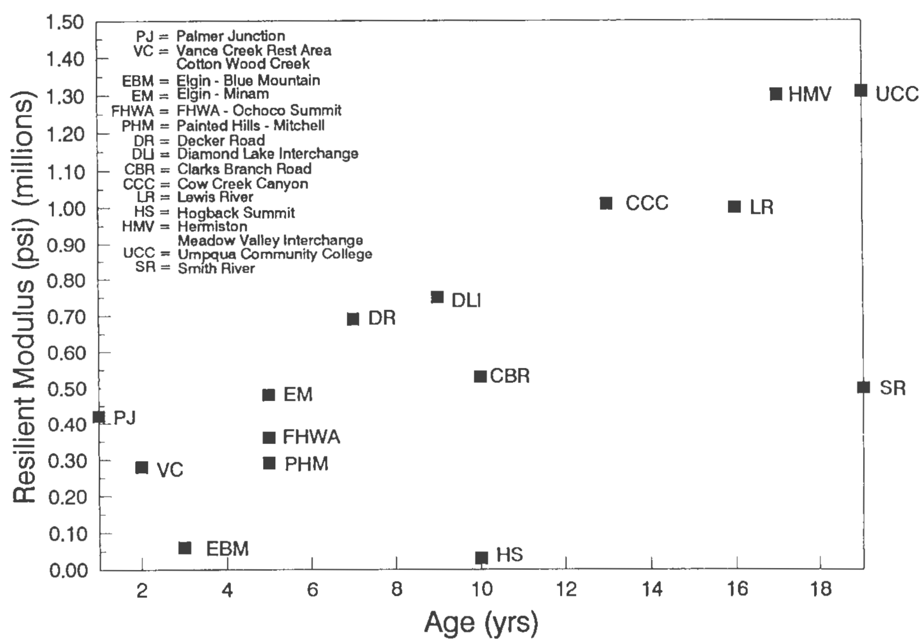


FIGURE 1 Variation in modulus with pavement age.

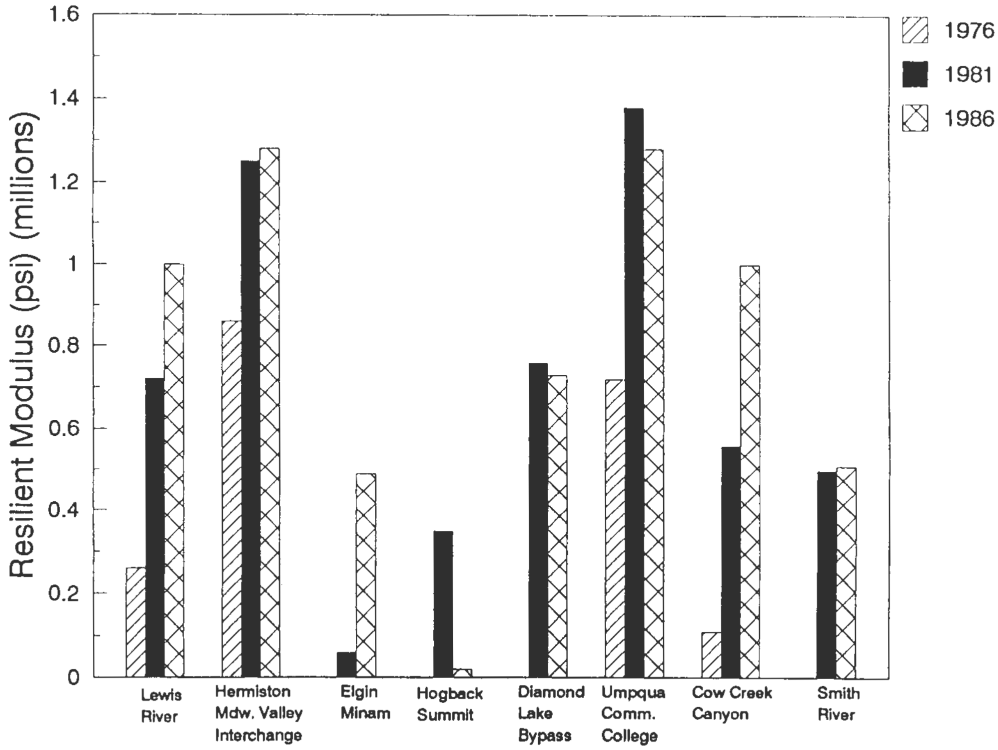


FIGURE 2 Variation in modulus with time, selected projects.

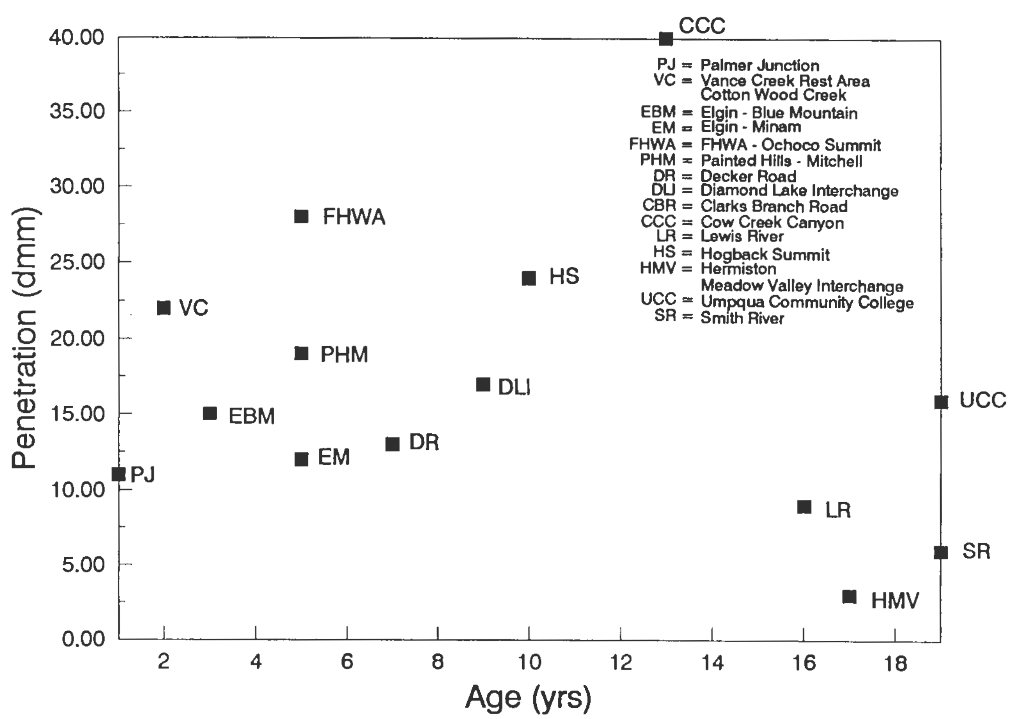


FIGURE 3 Variation in penetration with pavement age.

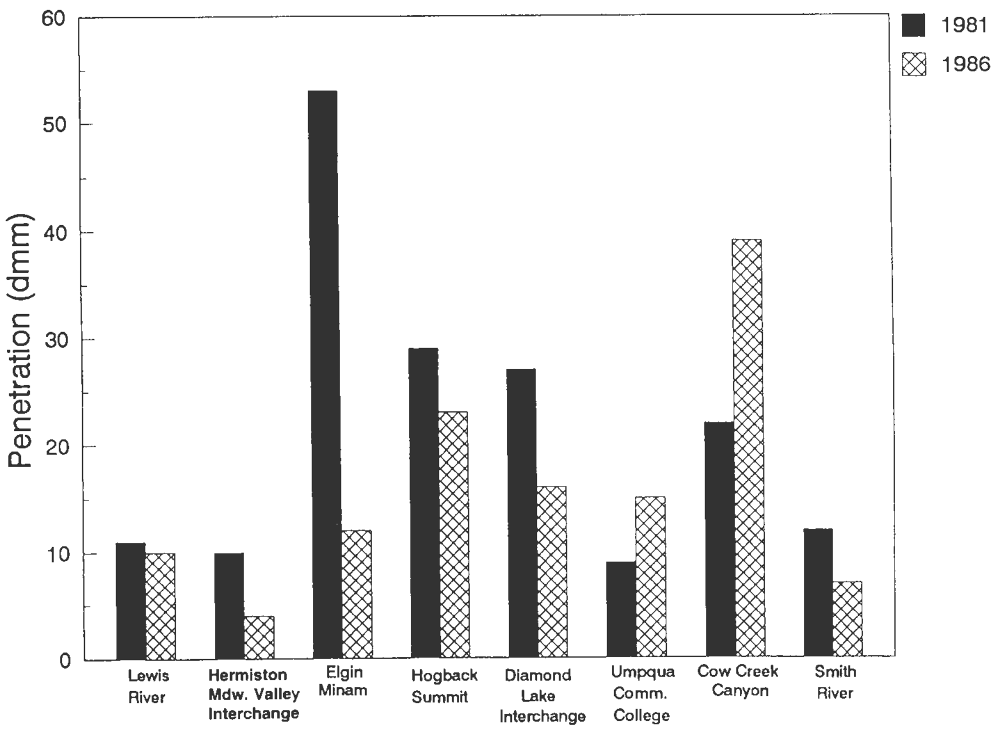


FIGURE 4 Variation in penetration with time, selected projects.

plotted in Figure 5. Cores with high penetrations have low mix modulus. However, the converse is not true. Several cores with low moduli also have low penetrations. It is interesting to note that projects with penetration values of 20 dmm or less tend to exhibit a higher incidence of cracking than those with penetrations over 20 dmm.

Structural Layer Coefficients

Calculations for layer coefficients using the approach described by Hicks et al. (*3*) are included in Table 4. Basically, the layer coefficients (a_1) are back-calculated using the AASHTO design guide (*7*) for $Pt = 2.0$ and a knowledge of the surface thickness (D_1), traffic (W_{18}), regional factor (R), and the soil support of the base (SS). The a_1 values are, in most cases, minimum values since the majority of open-graded emulsified asphalt pavements are still performing well. The values shown below for selected projects are based on some evidence of distress (thermal cracking, load-associated cracking, or overlaid):

Project	a_1-*Values*
Lewis River	0.37–0.41
Hermiston-Meadow Valley Interchange	0.45
Logan Valley	0.48
Hogback Summit	0.47
Diamond Lake Bypass	0.57
Cow Creek Canyon	0.75
Mapleton	0.21

For the projects surveyed, it appears that an a_1-value of 0.40 or greater is reasonable for open-graded emulsified asphalt pavements. Average values of layer coefficient for materials used in the AASHTO road test include an a_1-value of 0.44 for dense asphalt concrete surface course. Hence, a layer equivalency of 1 seems justified for open-graded emulsion mixtures. This layer equivalency is presently used by the Oregon and Washington DOTs (*8*,*9*). In fact, in some cases, Washington DOT actually uses a layer equivalency of less than 1.0 for OGEM because of the excellent performance of the OGEM pavements.

Guidelines for OGEM Use

The results in this paper clearly indicate that OGEM pavements can perform well for up to 25 years. This is, in part, because OGEM has been used only for selected applications. The general guidelines in Table 5 appear

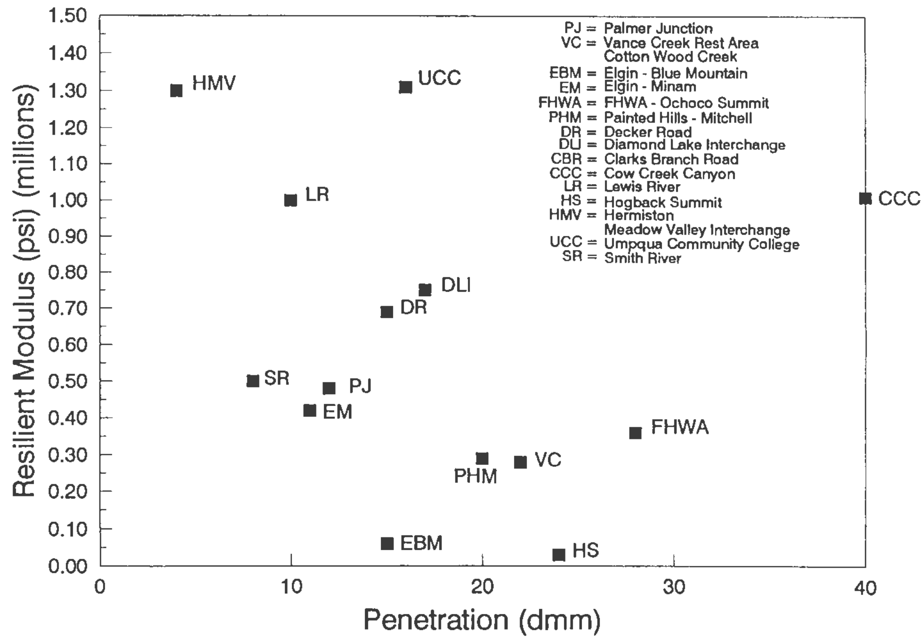


FIGURE 5 **Mix modulus versus asphalt penetration.**

to be appropriate for OGEM use in the Pacific Northwest (*4*). Specific guidelines by agency follow.

Oregon DOT Guidelines

Guidelines currently used by Oregon's DOT are summarized below.

1. Project selection

- Use in warmer, drier climates to facilitate curing.
- Use on projects with less than 5,000 ADT or less than 10 million ESALs.
- Use on higher-volume routes when traffic can be kept off pavement during curing.
- Use on west side where pavement flexibility is design criterion.

2. Thickness design

- OGEM possesses lower stiffness and higher fatigue resistance than hot-mix asphalt.
- Use 25 percent higher fatigue strength in mechanistic analysis. This results in a thinner OGEM layer than hot-mix asphalt but a thicker base.
- Overlay design based on tolerable deflection chart developed by Oregon State University (*4*).

3. Specifications for materials and construction

- Use CMS-2 or HFMS-2. CMS-2S can also be elected by contractor for better paving characteristics and ride and/or with dirtier aggregate.
- Fracture, gradation, aggregate quality, as specified in Section 735 in Oregon DOT specifications.
- Chip seal (6 to 2 mm typically) required on all projects.
- Mix design checks for compatibility of rock and emulsion. Emulsion content and add water are given as guide for field. Actual emulsion content used in field based on visual assessment of percentage of greybacks (all the emulsion it will hold without runoff).

In addition, it has been determined that OGEM has certain advantages and disadvantages.

1. Advantages

- Cost—one-half to two-thirds that of hot ACP.
- Uses less energy and resources. Yields more pavement per ton mix.
- Simpler system. Less equipment needed.
- Allows use of very soft asphalt that improves resistance to thermal cracking and fatigue cracking.
- Resists rutting due to high fracture and open grading.
- Not sensitive to construction variability.

2. Disadvantages

- Long curing period required—limits use under heavy traffic.

TABLE 4 Backcalculated Layer Coefficients for Selected OGEM Projects

Project	Date Constructed	Pavement Condition	Traffic — W_{18}		Thickness of Open-Graded Pavement (D_1) (cm)	Estimated Regional Factor (R)	Average Base Property			$SN = f(W_{18},SS,R)$ $P_t = 2.0$	Calculated $a_1 = SN/D_1$
			1981	1986			CBR	R-Value	Soil Support (SS)		
Lewis River	1970	fair-good	$3.9x10^6$	$4.0x10^6$	21.6	2.0	—	60	6.5	3.3	.37-.41
Hermiston-Meadow Valley Interchange	1969	fair	$0.15x10^6$	$0.27x10^6$	10.2	2.0	—	70	7.0	1.8	0.45
Logan Valley	1971	good	$0.33x10^6$	$0.39x10^6$	10.2	2.0	31	—	7.5	1.9	0.48
Hogback Summit	1976	very good	40,000	52,500	7.6	2.0	—	70	7.0	1.4	0.47
Diamond Lake Bypass	1977	very good	88,500	$0.12x10^6$	7.6	2.0	—	70	7.0	1.7	0.57
Indian Caves-Medicine Creek	1971	good	5000	7200	20.3	2.0	—	—	—	—	—
Umpqua Community College	1967	very good	10,000	$0.31x10^6$	7.6	2.0	—	—	—	—	—
Cow Creek Canyon	1973	very good	$0.99x10^6$	$1.04x10^6$	7.6	2.0	31	—	7.5	2.25	0.75
Smith River	1967	good	$2.4x10^6$	$2.5x10^6$	25.4	2.0	—	—	—	—	—
Mapleton	1978	good	27,000	$0.10x10^6$	10.1	2.0	—	70	7.0	1.6	0.21

Note: 1 in. = 2.54 cm

TABLE 5 Suggested Guidelines for Use of OGEM

1.	ADT ≤ 5000.
2.	Do not use where sharp turns are expected.
3.	Complete paving during warm weather to insure curing.
4.	A positive seal is needed, preferably on the surface, to protect the underlying base from moisture and the surface from raveling.
5.	OGEM is useful on projects requiring good crack resistance.
6.	OGEM should not be used where high initial stiffness is required.
7.	Encourage use in locations where hot-mix plants do not exist.
8.	Encourage use where good aggregates are in short supply.

- Harder to get smooth ride due to variability of mix under screed.

Finally, the following are some general notes regarding the use of OGEM:

1. Use on Interstate for inside passing lane where cure time is adequate. Currently working with asphalt suppliers to develop a controlled set emulsion that can be used to obtain a shorter curing period and OGEM can be used under higher traffic volumes.
2. Need to experiment with OGEM without chip seals to use benefits of porous pavement.
3. Need to experiment with OGEM as porous overlay on portland cement concrete (PCC) pavement.
4. Vision for future is that OGEM will become a predominant mix in Oregon as emulsion technology improves so it can be used in cooler and wetter climates and under heavier traffic volumes.
5. Need to experiment with using OGEM for shoulders with PCC pavements.

Washington DOT Guidelines

General guidelines currently used by Washington's DOT include the following:

1. Project selection

- Use only in eastern Washington where dry climate facilitates curing.
- Use on routes with ADTs between 1,500 and 5,000.
- Use exclusively in more remote areas where portable ACP plants would be required.
- Do not use in urban areas where numerous turning movements and more critical tracking problems would occur.
- Do not use on projects where signficant grading work during the summer makes latter season paving necessary.

2. Specification and construction concerns

- Washington continues to use the federal guideline for grading and fracture. The grading of the course material does not appear to be very critical but the grading of the fine material is highly critical. Problems with early emulsion break and subsequent rough ride occur when the percentage of passing the 0.074 mm (No. 200) sieve approaches or exceeds 2.
- Washington has also had recurring problems with CMS-2 and CMS-2S emulsions from some suppliers: emulsions break early apparently from a deficiency of oil distillate. Washington now uses a modified CMS-2 specification for OGEM that requires 6 to 12 percent oil distillate by volume of emulsion, which has significantly reduced the early break problem. The standard CMS-2 specification is also modified to require a minimum viscosity that helps to eliminate runoff problems.
- All OGEM projects are fog-sealed or chip-sealed after construction to minimize potential raveling of the surface.

FHWA Guidelines

General guidelines currently used by FHWA include the following:

1. Project selection

- Use where very heavy loads are anticipated.
- Use where anticipated deflections may exceed 0.9 mm (0.035 in.).
- Use where marginal aggregates are the only economically available materials.
- Use over existing pavement structures that are flushed but are structurally adequate.
- Use where there is not an abundance of quality aggregate since a ton of materials yields more pavement structure.

2. Specification concerns

- Variability of emulsion due to manufacturing techniques.
- Specifications do not define the quality of the product.
- Specifications are too broad and allow product variability.
- FHWA specification spells out the minimum amount of oil distillate. Probably should use boiling point for the oil distillate to eliminate the slower curing materials.

Summary

This paper presents a summary of the experiences with the use of OGEM in the Pacific Northwest. OGEM has performed successfully in this region since 1966. Although some construction problems occurred early on, they have generally been resolved. The performance of these mixes has been very good to excellent, particularly east of the Cascades where the climate is dry and cold. OGEM continues to be a cost-effective pavement type for low-volume roads (1,000 to 5,000 ADT) used by FHWA, state highway agencies, and counties in the Pacific Northwest.

References

1. Richardson, E. S., and W. A. Liddle. *Experience in the Pacific Northwest with Open Graded Emulsified Asphalt Pavements.* Implementation Package 74-3. FHWA, U.S. Department of Transportation, 1984.
2. Hicks, R. G., and D. Hatch. Performance of Open Graded Emulsion Mixes, *Proc., 14th New Mexico Paving Conference*, 1977, pp. 184–212.
3. Hicks, R. G., et al. Performance Evaluation of Open-Graded Emulsified Asphalt Pavements. *Proc. AAPT*, 1983, pp. 441–473.
4. Hicks, R. G., et al. Performance Update of Open-Graded Emulsified Asphalt Pavements in the Pacific Northwest. *Proc., Asphalt Emulsion Manufacturing Association*, 1988.
5. Mahoney, J. P., et al. *Mechanistic-Based Overlay Design Procedure for Washington State Flexible Pavements.* Report No. WA-RD 170.1. Washington Department of Transportation, 1989.
6. Tholen, O. *Field Investigation in Hammarstrand, Comparing Fatigue Strain in an Open-Graded Asphalt Emulsion Mix and a Dense Hot Mix.* Unpublished report. Sweden, 1989.
7. *AASHTO Guides for Design of Pavement Structures.* American Association of State Highway and Transportation Officials, 1981.
8. *Design Procedure for Flexible Pavements.* Oregon Department of Transportation, 1981.
9. *Design Procedure for Flexible Pavements.* Washington Department of Transportation, 1986.

BRIDGES

Modular Timber T-Beam Bridges for Low-Volume Roads

Barry Dickson, *West Virginia University*

Wood has a long and successful history as a bridge-building material. Recently, a new wood construction technique known as stress laminating has been developed sufficiently so that wood bridges may become cost-competitive with concrete and steel, particularly for short span ranges and on low-volume roads. At the Constructed Facilities Center of West Virginia University (WVU), a variation of the stress-laminated deck has been developed that shows excellent structural performance and reasonable costs. The modular timber T-beam bridge is the result of many years of effort on the part of engineers from WVU, West Virginia Division of Highways, the USDA Forest Service, and the USDA Forest Products Laboratory. Ten modular timber T-beam bridges are currently in service throughout West Virginia, and one is in service in the Ozark National Forest in Arkansas. Modular timber T-beam bridges consist of glued-laminated wood beams and wood deck planks stress-laminated together to form 4-ft-wide modules, each as long as the bridge span. To illustrate the advantages and problems of the modular system, two case studies are presented. The Camp Arrowhead bridge is one of the first modular T-beams built and is an example of a moderately priced structure showing excellent performance. The Nebo bridge, a short modular T-beam bridge, offers some insight into the construction problems resolved during the development process.

Timber has a long tradition as a bridge-building material for low-volume roads. From the earliest log structures to modern engineered timber bridges, wood was often selected by bridge builders for its strength and availability and because it could be easily worked and handled. In recent years, steel and concrete have replaced timber for much of our bridge construction because these materials are considered by most of today's engineers to be stiffer, stronger, and more durable than timber. New timber technology has created better timber products, however, and timber could once again compete with steel and concrete, particularly for short spans on low- and medium-volume roads.

Timber technology has evolved from dependence on solid sawed components to fabrication of components formed from many smaller pieces. Plywood, laminated veneer lumber, and oriented strand board are typical engineered timber components that combine smaller pieces of wood to form a strong, high-quality timber component. These new products are often less expensive and more readily available and have more reliable engineering properties.

A less recognized innovation in timber engineering emerged in Canada in the late 1970s: stress laminating. Used principally for bridge decks, stress laminating is a technique of pressing boards (usually 2 or 3 in. thick and 10 to 16 in. wide) together to form a bridge deck. The traditional fastening method for laminating adjacent planks—nailing—has a relatively short practical life and limited load-sharing capabilities. Stress laminating, which requires more expensive hardware and equipment, has been shown to dramatically improve the load-carrying capacity and durability of timber deck bridges (*1*).

Since 1988, a federally funded project has been promoting the use of timber for bridges in the United States. The Timber Bridge Initiative (TBI), administered by the USDA Forest Service, State and Private Forestry, provided funding for demonstration bridges in all 50 states and for research to improve timber bridge technology. In West Virginia, the West Virginia Department of Highways (WVDOH) and the Constructed Facilities Center (CFC) at West Virginia University joined forces to design and construct 60 modern timber bridges between 1988 and 1994, most of which were partially funded by the TBI. Several types of stress-laminated and glued-laminated bridges have been built, but perhaps the most innovative and efficient is the stress-laminated modular T-beam.

Early Stress-Laminated Timber Bridges in West Virginia

Stress-laminated timber decks (Figure 1) are a significant improvement over nail- or dowel-laminated decks, but span lengths are limited by the availability of large-dimension lumber. Butt-joined laminations are acceptable in stress-laminated decks, so long lengths are not required. To resist the bending stresses, however, depths up to 16 in. are often necessary. The practical upper limit for stress-laminated decks is 30 to 35 ft using the local hardwoods available in West Virginia. The first stress-laminated timber bridge planned for West Virginia had a span of 73 ft, far greater than any stress-laminated timber bridge previously built. To construct a stress-laminated timber bridge to cross that span, modifications to the existing designs were necessary to create added stiffness. The T-beam system, which originated at the CFC, combines glued-laminated beams with a stressed deck to create a substantially stiffer structure than the stressed deck alone (Figure 2).

West Virginia's first stress-laminated timber bridge, the Barlow Drive T-beam bridge, was constructed in May 1988 and has been serving the local community well since then. Approximately 500 vehicles, many of them heavily loaded trucks, use the bridge daily. A nearby concrete plant and an oil depot have been able to reduce the length of many trips that previously required a detour around the old steel truss bridge. However, at $79/ft^2, the new timber bridge was not competitive with those built with precast concrete.

Between 1988 and 1991, 30 more stress-laminated timber bridges were built in West Virginia. The success of the first T-beam bridge played a large role in the rapid expansion of the TBI but none of the next 30 bridges built was a T-beam. Rather than design each bridge individually, most of the 30 bridges were designed from standard plans prepared at the CFC and WVDOH. The first standard plans included stress-laminated decks and a modification of the T-beam, the stress-laminated box beam, but not stress-laminated T-beams. It was not until 1992 that stress-laminated T-beam standard plans were prepared and incorporated into the WVDOH standard plans.

After the first 2-year period of stressed timber bridge construction, it became apparent that all of the new timber bridge types cost more than expected. The least expensive system, the stress-laminated deck, cost an average of $42/ft^2 and the box-beam structures cost, on average, $60/ft^2 (*2*). Unfortunately, the least expensive timber bridges performed poorly (*3*) and would require either deeper (and more expensive) timber or added stiffeners to meet AASHTO performance requirements. Clearly, changes were necessary if timber bridges were to be competitive in an open market.

Modular Stress-Laminated Timber T-Beam Bridges

An "optimization" project was conducted at the CFC to determine what changes could be made to improve performance and decrease costs. With the cooperation of engineers from the USDA Forest Products Laboratory, WVDOH, and Burke-Parsons-Bowlby Corp. (the largest timber bridge fabricator in West Virginia), a critical review of the bridge systems already built was performed. A modular concept was proposed to reduce costs and improve performance. The modular T-beam system—which should be less expensive than a box beam (because it uses less material) and should perform better than the stress-laminated decks (because it is stiffer and stronger)—was selected as the system with the most potential to compete with precast concrete.

Advantages of Modular Systems

Numerous improvements were expected by changing to a modular bridge construction. Primarily, constructing the bridge in modules shifts most of the assembly operations to a fabrication shop where costs were expected to be less than at the bridge site and the quality of workmanship more consistent. Second, the cost of installing the modules was expected to be lower due to

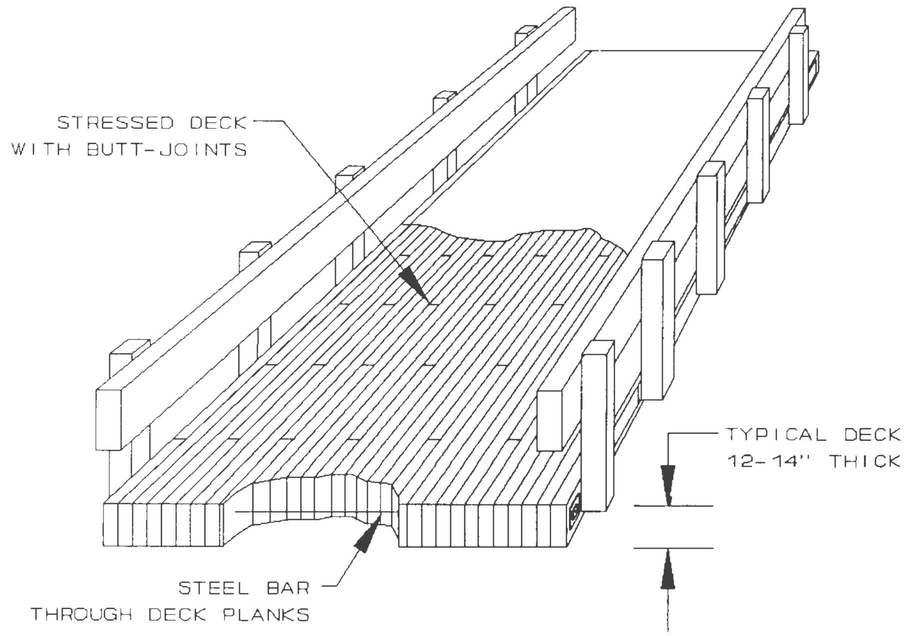


FIGURE 1 Stress-laminated deck.

the smaller-sized crane required and shorter installation time.

Design of Modular T-Beam

Although the current AASHTO specifications (*4*) do not yet contain design guides for stress-laminated T-beam structures, the similarity of stress-laminated decks and stress-laminated T-beams allows many of the provisions to apply (Figure 3). In many important phases of the design, however, the differences between the two types of bridges are substantial. For these areas of the design, the CFC has developed its own standards. These standards, which have been submitted to AASHTO for possible incorporation into the Standard Specifications, are based on laboratory and field testing, as well as on theoretical studies.

The design process used by the CFC is a relatively simple one. Each of the beams of the bridge is assumed to consist of a single-width web with a fully composite flange. The flange width is determined to have an "effective" width that is used in the calculation of the moment of inertia of the T-beam unit. Generally, the effective width of the flange is less than the spacing of the beams (Figure 4).

Each T-beam unit is designed to support a portion of the expected live load plus a share of the dead load. For highway bridges in West Virginia, an AASHTO HS-25 truck loading is required. Because each T-beam unit shares the loading with the other T-beam units, a load distribution factor can be applied. Depending upon the number of beams, the spacing of beams, and the stiffness of deck members joining the beams, the load distribution factor can reduce the applied load to one-half of the HS-25 wheel load.

After the effective flange width is determined, the dimension of the T-beam can be established. The stresses and deflection of the T-beam can then be found using the load distribution factor to calculate the live load that must be supported by the beam. The beam is modeled as being simply supported with a span equal to the bridge center-of-bearing to center-of-bearing spacing. Stress-bar spacing and size, bearing plate size, and stress-bar force level are then chosen in the manner prescribed in the Standard Specifications. The full design process can be found elsewhere (*5*).

The designs prepared at the CFC became the basis for a new set of standard plans for the WVDOH. The new stress-laminated modular T-beams have a span range of 24 to 63 ft using glued-laminated beams from 19.25 in. to 49.5 in. deep. Some of the shorter spans use a 7-in. deck, but most of the standard plans require a 9-in. deep northern red oak deck.

Fabrication of Modular T-Beam

Each module of the stress-laminated modular T-beam bridge consists of two glued-laminated beams and 30 deck planks (the number of deck planks can vary slightly to create wider or narrower modules). To fabricate an exterior module, one 10-in.-wide glued-laminated beam, one 5-in.-wide glued-laminated beam,

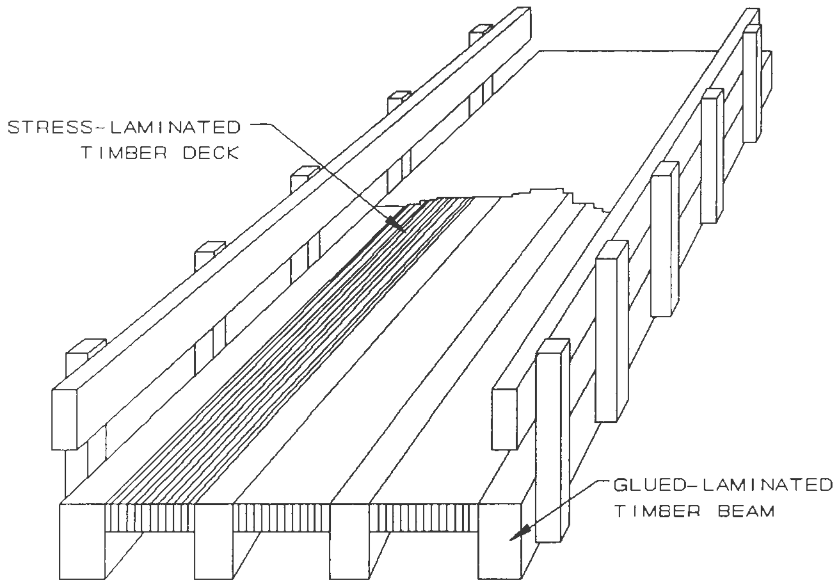


FIGURE 2 Stress-laminated T-beam.

and thirty 1.5-in.-wide boards are joined by high-strength steel stressing bars. One set of high-strength steel bars passes through holes drilled in the planks and the beams on 2-ft centers (temporary bars), and another set of bars passes through holes on 6-ft centers (fabrication bars). Interior modules are constructed similarly except both beams of the interior modules are 5 in. wide.

Both interior and exterior modules are stressed three or more times at the fabrication shop using both the temporary bars and the fabrication bars. Guide rail posts, curbs, and guide rails are fastened to the exterior modules. After approximately 6 weeks in the fabrication shop, the modules are ready to be shipped to the bridge site.

Erection of Modular T-Beam

Generally, one or two modules are shipped to the bridge site on each truck. Just before the modules are craned onto the abutments, all the 2-ft center bars are removed leaving only the fabrication bars to maintain the compressive force on the modules. Once all the modules are positioned on the abutments, full-length steel bars are inserted through the vacant holes on 2-ft centers and the entire bridge is stressed one final time. The steel bars on 6-ft centers in the exterior modules can then be removed for reuse on another bridge; the fabrication bars in the interior modules remain in the bridge.

Because the guide rail and curb have already been fastened to the exterior beams, all that remains to com-

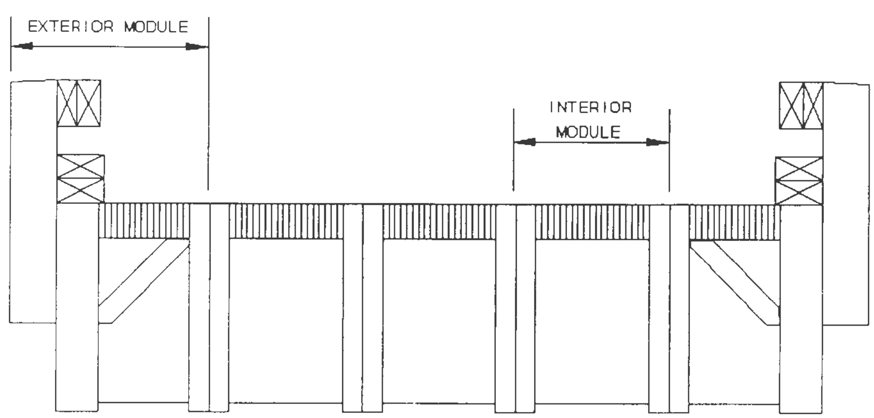


FIGURE 3 Cross section of typical modular T-beam bridge.

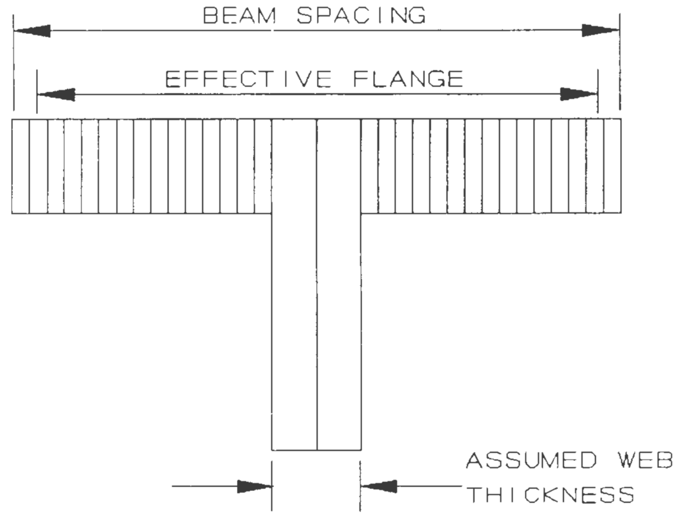


FIGURE 4 Design T-beam unit.

plete the bridge is to cast the concrete back walls, erect the approach rails, complete the approach subgrade, and then pave. The erection of the modules—if all steps go as planned—should take less than 1 day.

Case Studies

Ten modular T-beam bridges were built in West Virginia in 1992 and 1993. Twelve more were in various stages of construction in 1994. Although these bridges have not been without problems, the general consensus has been that structures perform better and are slightly less expensive than their predecessors. Two bridges have been selected as case studies for this report; the Camp Arrowhead bridge in Cabell County, West Virginia (Case A), and the Nebo bridge in Clay County, West Virginia (Case B). Technical data for these bridges are shown in Tables 1 and 2. These two bridges are representative of all the modular T-beam bridges already built. The Camp Arrowhead project proceeded as planned, with only a few delays. The Nebo bridge project was not as smooth and was delayed several weeks, but the bridge is now in service and functioning well. By studying these cases and the other bridges built, specification and design revisions can be made as needed. But, as these cases show, the costs of stress-laminated timber bridges remain high.

Case Study A: Camp Arrowhead Bridge

The new timber bridge crossing Little Cabell Creek near the Camp Arrowhead Boy Scout Camp replaces a precast concrete bridge on timber pilings (Figure 5). The concrete bridge had a span of only 22 ft, almost 40 ft shorter than the new timber span. The extra span length is a result of the use of spill-through abutments. The spill-through abutments used in West Virginia are stub abutments generally set back 15 to 20 ft from the normal stream channel to prevent scour of the foundation. The low height of the spill-through abutments also reduces the volume of earth retained, thus increasing the longevity of the abutment.

The stress-laminated, modular T-beam bridge was fabricated by Burke-Parsons-Bowlby Corp. in Spencer, West Virginia, and erected on May 12, 1988, by WVDOH forces. Because this bridge was the first West Virginia bridge built using the new modular construction technique, a great deal of attention was given to the fabrication, erection, and performance.

The fabrication of this modular T-beam was significantly different from the procedures previously used. Five modules, each 63.5 ft long, were manufactured at Burke-Parsons-Bowlby Corp. The two exterior modules are each 60 in. wide, and the three interior modules are 55 in. wide. Following the WVDOH specifications (6), each of the modules was stressed two times over a period of 6 weeks. As the modules were stressed, it was discovered that some of the modules were becoming misshapen (Figure 6). Several attempts were made to remedy the problem. Temporary steel diaphragms were bolted to the beams before stressing to maintain squareness, but they were ineffectual because the lag bolts used to fasten the diaphragms to the beams would pull out as the stressing was applied. An additional set of steel bars was installed at the bottom of the beams to pull the bottoms together as the stress was applied.

TABLE 1 Technical Data for Camp Arrowhead Bridge

TECHNICAL INFORMATION

Bridge Type: Modular T-BEAM
Abutment Type:Stub on piles
Design Load: HS-25
Average Daily Traffic: 50

Geometry

Number of Spans: 1
Out-to-Out Length: 63'-6"
Center of Bearing-to-Center of Bearing: 62'-0"
Number of Lanes: 2
Out-to-Out Width: 23'-9"
Curb-to-Curb Width: 21'-9"
Skew: NONE
Beam Size: 10"x44", 5"x44"
Deck Depth: 9"

MATERIALS INFORMATION

Deck Lumber

Grade: 3 or better
Species: NORTHERN RED OAK
Quantity: 11,250 bf
Sizes Used: 1 1/2"x9"

Beams

Grade: 24FV3
Species: SOUTHERN PINE
Quantity: 14,000 bf

Guiderail system

Posts: NONE
Size:
Grade:
Species:

Curb

Size: 6"x12"
Grade: 3
Species: NORTHERN RED OAK

Rail

Size: NONE
Grade:
Species:
Quantities:
Wood:
Steel:

Preservative

Type: COAL-TAR CREOSOTE
Quantity: 22,800 lbs.

Steel

Bar Size: 5/8"
No. of Bars: 32 full length
50 fabrication
Plate Size: 7"x11"
Quantity: 3500 lbs.

COSTS

Fabrication Cost: $79,512
Erection Cost: $10,500
Substructure Cost: n/a
Total Project Cost: $90,012

Again, the results were less than successful. After one beam cracked near midheight, this method was also abandoned. The solution chosen by the manufacturer was to remove the stressing bars, insert a tapered deck board, and restress the misshaped module. Although this method does not address the cause of the problem, it did result in a module with minimal deviation from the desired shape. Upon completion of the specified stressing sequence, the curb was installed to the two outside modules (because this bridge is on a very low-volume and low-speed road, no rail was needed).

Each module was loaded onto a "stretched" trailer and trucked from the fabrication plant in Spencer to the bridge site in Cabell County, a distance of about 50 mi. A single 60-ton crane lifted the modules from the truck with the aid of a lifting system designed specifically for the modular bridges (Figure 7). A second crane was used to help position the modules. The lifting system is composed of steel eyebolts through the deck connected to two lightweight steel angles under the beams. The angle steel is slightly shorter than module width so that the modules can be set on the bridge seats tightly against the neighboring module.

The WVDOH District 2 crew installed the bridge in less than 2 days. After the modules were craned onto the bridge seats, the full-length bars were inserted and stressed and the exterior modules' fabrication bars were removed. No problems were encountered with the installation or the stressing. To erect the bridge and stress the modules required 130 man-hours; the cranes were at the site for 30 hr but were used for approximately 15 hr. The back walls were cast before the installation of the timber modules, which left only approach work and paving to be done before the bridge could be

TABLE 2 Technical Data for Nebo Bridge

TECHNICAL INFORMATION

Bridge Type: Modular T-BEAM
Abutment Type: Stub on piles
Design Load: HS-25
Average Daily Traffic: 250

Geometry

Number of Spans: 1
Out-to-Out Length: 33'-6"
Center of Bearing-to-Center of Bearing: 32'-0"
Number of Lanes: 2
Out-to-Out Width: 21'-3"
Curb-to-Curb Width: 19'-5"
Skew: NONE
Beam Size: 10"x27", 5"x27"
Deck Depth: 9"

MATERIALS INFORMATION

Deck Lumber

Grade: 3 or better
Species: NORTHERN RED OAK
Quantity: 3,950 bf
Sizes Used: 1 1/2"x9"

Beams

Grade: 24FV3
Species: SOUTHERN PINE
Quantity: 4,550 bf

Guiderail system

Posts: 20
Size: 12"x8"
Grade: 3
Species: NORTHERN RED OAK

Curb

Size: 5"x10"
Grade: 24FV3
Species: SOUTHERN PINE

Rail

Size: 14"
Grade: A36
Species: STEEL
Quantities: 9500 lbs.

Preservative

Type: COAL-TAR CREOSOTE
Quantity: 8,275 lbs.

Steel

Bar Size: 5/8"
No. of Bars: 17 full length
25 fabrication
Plate Size: 8"x14"
Quantity: 1150 lbs.

COSTS

Fabrication Cost: $45,200
Erection Cost: $9,913
Substructure Cost: n/a
Total Project Cost: $55,113

opened for traffic. The bridge was not paved for several months after opening, but stress-laminated structures do not need to be paved to be driven on temporarily.

Testing of the Camp Arrowhead bridge began soon after completion of construction. A series of load tests, bar-force monitoring, moisture content measurements, and elevation measurements has been performed as part of a cooperative USDA Forest Products Laboratory and CFC project. Although the testing program is still in progress, the bridge has shown good performance in most categories. The stiffness of the structure, as measured by a live-load test, is slightly higher than anticipated. Moisture contents are near the specified levels and the bridge is maintaining camber. Bar forces are dropped more rapidly than expected, but they have stabilized at an acceptable level. The creosote retention of the bridge beams is vastly improved when compared with the 1989 funded bridges, but some bleeding still occurs during the hot summer months.

The cost of the Camp Arrowhead bridge delivered to the site was $79,512. Because WVDOH forces erected the bridge, the total costs of the completed structure are not known as precisely as they would be if a contractor had bid the project. Based on the time records of the state crew and using an estimated hourly rate of $20 for labor and $1,500 per day for crane time, the cost of the bridge installation was approximately $10,500. The cost per square foot of bridge surface area was $54.

Case Study B: Nebo Bridge

The Nebo bridge was the third stress-laminated modular T-beam bridge constructed in West Virginia (Figure

FIGURE 5 Camp Arrowhead bridge.

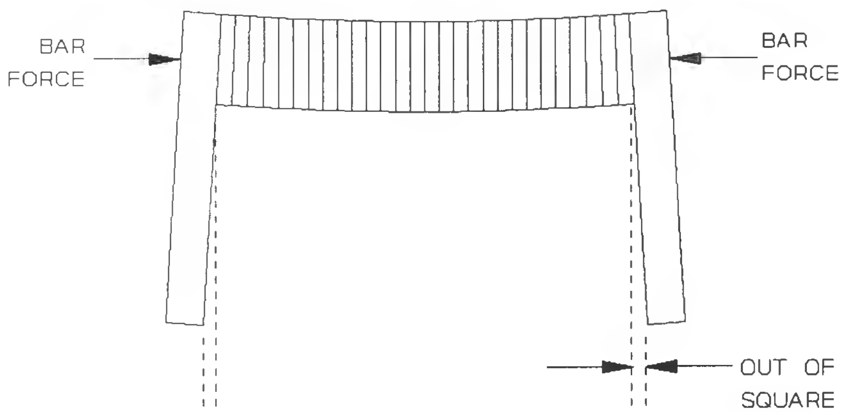


FIGURE 6 Misshapen module.

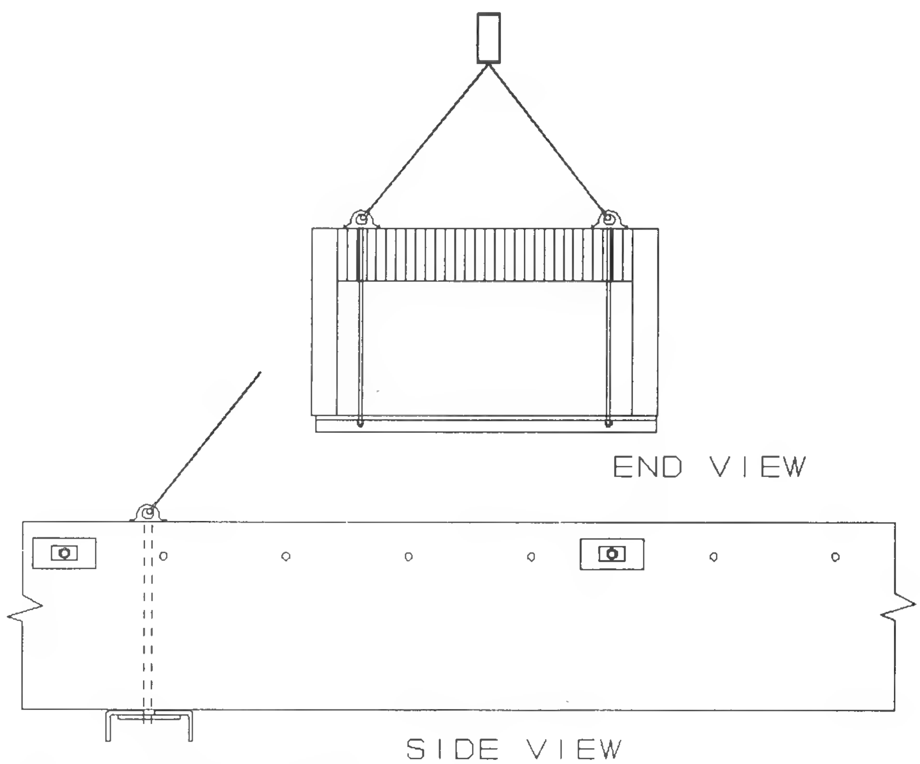


FIGURE 7 Module lifting system.

FIGURE 8 Nebo bridge.

8). The bridge is located in Clay County, West Virginia, and carries about 250 vehicles per day over the Stinson Creek. The 33-ft-long bridge rests on spill-through abutments on steel piles.

Fabrication of the Nebo bridge was done by Burke-Parsons-Bowlby at its Spencer plant. Like the other stress-laminated modular T-beam bridges, the fabrication sequence followed the procedure prescribed by the WVDOH specifications. And, like the first two modular T-beam structures, the stressing operations caused the modules to distort. On this bridge, however, no tapered boards were used to correct the problem based on the supposition that the modules would assume the expected rectangular shape after the final stressing at the bridge site.

Unfortunately for the contractor, the bridge did not take the expected shape after installation on the abutments and stressing. Rather, the error that appeared relatively minor on each module accumulated when the entire bridge was assembled. After stressing, the exterior beams lifted off the abutments and the structure took a noticeably distorted configuration.

Several potential solutions to this problem were considered. After a brief test run at the fabrication shop, engineers from WVDOH and CFC decided that the best option was to install a set of steel stressing bars through all the modules immediately beneath the deck. Located at this position, very little bending of the glued-laminated beam would result from the tensioned rods and maximum compressive force would be applied where it was needed.

The repair was successful and relatively inexpensive. Testing and inspection of the Nebo bridge have shown no adverse effects from the repair procedure. To prevent recurrence of this problem, WVDOH specifications have been modified to require that the modules be fabricated to within plus or minus 0.25 inch of the specified dimension. By applying a more uniform compressive force, the fabricators have been able to comply with the new specification. Only minor installation problems have been observed in the more recently built bridges.

Live-load testing and periodic monitoring of the bar force levels and moisture levels have shown the Nebo bridge to be performing well. The deflection of the bridge at the centerline was 0.375 inch when loaded with a 50,000-lb vehicle. Bar-force levels remain above 50 percent of the applied force and moisture levels are near 20 percent. Creosote retention of the beams and decks has been excellent.

The cost of the Nebo bridge, delivered to the site, was $45,200. The contractor charged the state $55,113 for the installed structure; thus the installation cost can be assumed to be the difference, $9,913. The cost per square foot of bridge surface area was $79.

REFERENCES

1. Taylor, R. J., B. DeV. Batchelor, and K. Van Dalen. *Prestressed Wood Bridges.* Report SRR-83-01. Ontario Ministry of Transportation and Communications, Toronto, Canada, 1982.
2. Hawkins, J. E., and C. Hassler. *Investigating Cost and Installation Time Data for Timber Bridges.* USDA-Forest Service, Northeastern Region State and Private Forestry, Morgantown, W.Va., 1991.
3. Gangarao, H. V., B. Dickson, and P. Raju. *Monitoring of Stress-Laminated Bridges.* CFC Report 91-156. Constructed Facilities Center, West Virginia University, Morgantown, 1991.
4. *Standard Specifications for Highway Bridges*, 13th ed. American Association of State Highway and Transportation Officials, Washington, D.C., 1983.
5. Davalos, J., and H. Salim. *Design of Stress-Laminated T-System Timber Bridges.* Timber Bridge Information Resource Center, USDA-Forest Service, Northeastern Region, Morgantown, W.Va., 1993.
6. *Standard Specifications for Roads and Bridges.* West Virginia Department of Highways, Charleston, 1986.

Portable Glulam Timber Bridge Design for Low-Volume Forest Roads

S.E. Taylor, K.P. Keliher, and J.D. Thompson, *Auburn University*
M.A. Ritter, *Forest Products Laboratory, USDA Forest Service*
G.L. Murphy, *USDA Forest Service, Atlanta, Georgia*

Improved stream crossings are needed to reduce construction and maintenance costs and reduce the environmental impacts from low-volume forest roads and skid trails. New designs of timber bridges are cost-effective alternatives for portable stream crossing structures. This paper discusses design criteria for portable timber bridges and presents one design for a portable, longitudinal glued-laminated (glulam) deck timber bridge. Design criteria for portable bridges generally should not be as conservative as those used in the design of permanent highway bridges. The longitudinal glulam deck bridge has performed well in service, and load test results demonstrate that highway bridge design procedures are conservative for portable bridge systems.

The development, harvesting, and maintenance of U.S. forest resources require an extensive roadway network over a wide spectrum of geographical conditions. In general, these roads are designed for low-volume traffic conditions and are often single lane and unpaved. Because forest management activities are both diverse and sporadic, traffic volumes and loads can vary significantly. During resource development and maintenance periods, traffic volumes are low and consist primarily of light passenger vehicles. However, during harvesting operations, roadways may be subjected to higher-volume truck traffic with loads in excess of the maximum legal highway load. In either case, roadway use is commonly limited to short periods over a relatively long forest management period. For example, roadway access may be required for only a 6-month period over a 10-year cycle. As a result, there is a trend to close these roads when they are not needed for management activities.

Forest roads typically require a large number of structures to cross streams and other topographical features. Rothwell (*1*) and Swift (*2*), in separate studies on forest roads and skid trails, found that stream crossings were the most frequent sources of erosion and sediment introduction into streams. Fords and corrugated-metal or concrete culverts have been common stream crossing structures on forest roads for many years. Using fords may introduce sediment into the stream as vehicles drive across. While culverts alleviate this problem, considerable sediment loads appear to be introduced into the stream during the excavation and fill work that accompanies culvert installation. Results reported by Swift (*2*) show that the cumulative amount of soil placed in a stream at the road-stream crossing during the construction period was over 10 times greater than the sedimentation during logging operations. In addition, culverts may clog with debris and then be washed out during heavy runoff periods, thereby introducing additional sediment into the stream. In the case of roads or trails that are not permanent, the stream crossing structure may be removed after logging operations or other activities are complete. Removal of a culvert also appears to introduce heavy sediment loads into the stream.

Historically, bridges for low-volume forest roads have been of two types: temporary or permanent. A common temporary bridge has been the log stringer bridge that is either removed or left to deteriorate at the end of the use period. The use of temporary log stringer bridges has substantially declined over the last decade because it has become increasingly difficult to locate logs of the size and quality required for bridge construction. In addition, if the temporary bridge is not installed or removed properly, there may be adverse impacts to water quality. Permanent bridges, which are constructed of wood, steel, or concrete, depending on span requirements and economic considerations, are typically designed for service lives of 40 to 50 years. These permanent bridges are not economically feasible for short periods and often require expensive maintenance for continued service. In addition, permanent bridges for limited-use, low-volume forest roads are commonly designed to a lower standard than most public access facilities and can be a potential liability to the bridge owner if public access is possible.

One potential solution to short-term bridge needs on low-volume forest roads is the concept of portable bridges. If properly designed and constructed, portable bridges can be easily transported, installed, and removed for reuse at multiple sites. This ability to serve multiple installations makes them much more economically feasible than a permanent structure. In addition, if they are installed and removed so that disturbance to the site is minimized, they can alleviate many potential water quality problems.

Many of the advantages of timber bridges, which include using locally available materials, having long service lives, being relatively lightweight and easy to fabricate, and being prefabricated, make them ideal for temporary stream crossings. The objectives of this paper are to discuss design criteria for portable bridges and review the design and performance of a portable longitudinal glued-laminated (glulam) deck timber bridge.

Background

A variety of temporary or portable bridge designs have been constructed from steel, concrete, and timber. The following paragraphs briefly summarize these concepts.

Steel Bridges

Temporary steel bridge designs include modular steel girder bridges, hinged steel bridges, railroad flatcars, bridges made of steel truss panels (similar to the military's Bailey bridges), pipe fascine systems, and trailer- or armored military vehicle-launched bridges. The spans for these steel bridges range from 5 to 75 m (15 to 250 ft). Of these designs, the most common are the modular girder and the hinged steel bridges.

Two examples of portable modular steel girder bridges are the EZ Bridge sold by Hamilton Construction Company of Springfield, Oregon, and those sold by Big R Manufacturing Company, Inc., of Greeley, Colorado. These bridges have modular sections constructed of steel I-beams that run longitudinally under a transverse steel or timber deck. The bridges typically come in two sections and are bolted together when installed. These bridges can be designed to meet vehicle loads specified by AASHTO or the U.S. Department of Agriculture (USDA) Forest Service. Since these bridges are prefabricated, they can be installed quickly. Forestry equipment can be used to install shorter span bridges; however, heavy construction equipment is required for long span bridges.

Another commercially available portable steel bridge design has been used successfully on many low-volume roads and logging operations. These bridges, which are manufactured by ADM Welding and Fabrication in Pennsylvania, are constructed of steel stringers with a timber deck. One of the smaller designs is 3.5 m (11 ft) wide and 7.9 m (25 ft) long and can be constructed to support either log skidder or truck traffic. Other bridges of this type have been constructed for spans up to 16.8 m (55 ft). These bridges have a unique hinged design that allows them to be folded in half and thus meet the legal width limit for highway transport. Bridges classified for log skidder and truck loads are advertised with capacities of 13 600 kg (30,000 lb) and 45 400 kg (100,000 lb), respectively.

Concrete Bridges

Alt (*3*) discussed the use of a portable prestressed concrete bridge for logging operations. A forest products company in Florida used this bridge for log truck traffic. It was constructed with three reinforced concrete slabs 1.2 m (4 ft) wide, 381 mm (15 in) deep, and 10.7 m (35 ft) long. Although this bridge was very cost-effective, its weight of 35 400 kg (78,000 lb) made it necessary to use heavy construction equipment for installation and removal. Therefore, this design is probably not suitable as a portable bridge for forest operations.

Timber Bridges

Timber bridge designs include log stringer bridges, timber mats or "dragline" mats, modular timber truss bridges, longitudinal stringer with transverse deck bridges, and longitudinal glulam or stress-laminated deck bridges. The difficulties with log stringer bridges were mentioned earlier. Many loggers use timber mats; however, most are not engineered products and are not advertised as bridge components by their manufacurers. Although log stringer bridges and timber mats have been used successfully for many years, the recent advances in timber bridge technology include several engineered designs that can be easily adapted for use as portable bridges.

Probably the most promising designs for spans up to 12 m (40 ft) consist of longitudinal glulam or stress-laminated decks that are placed across the stream. These longitudinal deck designs are relatively simple to construct, are somewhat lightweight, and have comparatively thin cross sections. They can be prefabricated into large sections that can be quickly and easily installed at the stream crossing site with typical forestry equipment, such as hydraulic knuckleboom loaders or skidders. Also, it may be possible to install these bridges without operating the equipment in the stream, which minimizes the site disturbance and associated erosion and sediment load on the stream.

Hassler et al. (*4*) discussed the design and performance of a portable longitudinal stress-laminated deck bridge for truck traffic on logging roads. This bridge was constructed of untreated green mixed hardwoods. It was 4.8 m (16 ft) wide, 12.2 m (40 ft) long, 54 mm (10 in) thick, and was fabricated in two 2.4-m (8-ft) wide modules. The bridge was installed to assist timber harvesting activities on the West Virginia University Forest and was placed directly on the existing stream banks without abutments. The bridge was installed with a typical hydraulic knuckleboom loader and a skidder and performed satisfactorily under load tests. No significant water quality changes occurred as a result of the bridge installation.

Taylor and Murphy (*5*) presented another design of a portable stress-laminated timber bridge. This bridge was designed for logging truck traffic and consisted of two separate stress-laminated panels 1.4 m (4.5 ft) wide. The panels could be constructed in lengths up to 9.7 m (32 ft). Each panel was designed to be stressed separately and then placed adjacent to the other panel with a 0.6 m (2 ft) space between panels. The overall width of the complete bridge was 3.3 m (11 ft). The deck panels could be placed on a mud sill that would sit directly on the stream bank. Curb rails ran the length of the bridge. This bridge has not yet been tested; however, various companies are currently fabricating similar portable stress-laminated timber bridges.

Design Criteria

General Considerations

Important characteristics that must be considered in the design and selection process for portable bridges include the design life, traffic type, and volume. The designer uses these characteristics to select the initial design concept and determine many important design criteria. For example, if the average daily traffic (ADT) is less than 50 vehicles per day and consists primarily of light vehicular traffic, it may be possible to use a curb instead of a full guardrail. Also, for many types of low-volume road bridges with short design lives, installing a wear surface on the bridge deck or using high levels of preservative treatments may not be necessary. However, if the bridge is expected to carry heavy off-highway vehicles, design loads must be accurately determined.

Table 1 gives examples of different design criteria that should be considered for three traffic volume categories: sub-low-volume, low-volume, and high-volume. The sub-low-volume road category might include skid trails and other temporary roads constructed during harvesting or management activities. These types of roads may be used by very light vehicles or by heavy off-highway vehicles. The low-volume road might include major haul roads carrying higher volumes of truck traffic. The high-volume roads consist primarily of public highways where temporary bridges are needed during construction or replacement of permanent bridges. The authors invite comments on these example criteria or suggestions for additional criteria.

A portable timber bridge should have several other general design characteristics to be a viable alternative for temporary stream crossings. The most important of these considerations may be the ease with which the bridge can be assembled, installed, removed, and transported. Ideally, field fabrication requirements should be kept to a minimum. The portable bridge design should facilitate installation and removal with typical forestry or light construction equipment. Many knuckleboom loaders or small cranes can lift bridge components weighing less than 3200 kg (7,000 lb) with lengths less than 10 m (32 ft). At sites where loader or crane use is not possible, forwarders or skidders can be used to drag bridge components to the stream and winch them into place. Regardless of the equipment used to place the bridge, provisions should be made to attach rigging materials to the components so they can be handled without damage. Also, it is important to design the bridge so that it can be transported on common log trucks or equipment trailers. Temporarily placing bridge sections on wheels and towing them to the site may be possible if roads are suitably constructed.

TABLE 1 Suggested Design Criteria for Portable Bridges Installed on Various Road Types

Criterion	Sub-Low Volume	Low Volume	High Volume
Design Life	≤ 5 years	≤ 15 years	> 20 years
Traffic Type	1. Off-highway 2. Light vehicles	1. Trucks 2. Light vehicles	1. Highway
Average Daily Traffic Flow	75	100	Unlimited
Design Speed	8 - 16 kph	8 - 16 kph	> 40 kph
Load Criterion	Off Highway Loads	Off Highway Loads or Highway Loads	AASHTO HS20 or HS25
Load Application Period	6 months	24 months	36 months
Deflection Criterion	None	None	AASHTO Criterion
Span Type	Simple	Simple	Simple
Span Length	< 10 m	≤ 10 m	≤ 25 m
Width	4 - 5 m	4 - 5 m	≤ 9 m
Rail	Curb or None	Rail or Curb	AASHTO Rail
Wear Surface	Wood or None	Wood or None	Asphalt

1 kph = 0.6 mph 1 m = 3.3 ft

Design Procedures

Design procedures for highway timber bridges can be found in the AASHTO standard specifications (*6*) and the publication by Ritter (*7*). Little previous research, however, has been conducted on appropriate design procedures for portable timber bridges on low-volume roads. Knab et al. (*8*) studied military theater-of-operations glulam bridges with design lives of 2 to 5 years. They concluded that using civilian design procedures, which are generally based on design lives of 50 to 75 years with relatively high levels of reliability, could result in unnecessarily conservative and uneconomical designs for the limited performance needs of temporary bridges. Using results from reliability analyses, they developed new design procedures and modification factors for allowable stresses that would result in adequate levels of structural safety for glulam girder bridges. They concluded that modification factors could be used to increase allowable bending, shear, and compression stresses for these temporary military bridges.

Other work by GangaRao and Zelina (*9*) examined the design specifications for low-volume civilian roads. They concluded that the use of urban highway standards for low-volume road bridges results in overly conservative and uneconomical designs. They defined low-volume roads as those with maximum two-directional average daily traffic (ADT) of 200 vehicles or maximum two-directional average daily truck traffic (ADTT) of approximately 30 trucks per day. They suggested that allowable stresses for steel and concrete structures might be increased for such roads and that deflection limits might be relaxed for steel bridges. They did not recommend changing the deflection criteria of $L/400$ and $L/300$, where L is the bridge span, for low-volume concrete or timber bridges, respectively.

These research results (*8,9*) indicate that applying AASHTO design procedures to portable bridges on low-volume roads may result in overly conservative designs. The designer must consider that, in many cases, the design life of such a bridge may only be 5 to 10 years. Therefore, it may be possible to make changes such as increasing the load duration factor above that currently specified by AASHTO. However, additional research is needed before suggesting other changes in design procedures.

Strength Criteria

The designer of a portable bridge must determine the applicable loads and load combinations for the specific situation. For portable timber bridges that will carry highway truck traffic, including logging trucks, AASHTO standard vehicle loads, such as the HS20-44 (HS20), should be sufficient in most cases. However, in situations where heavier off-highway vehicles are used, the USDA Forest Service uses several additional standard vehicle overloads, such as the U80 or U102 truck (7).

If the portable bridge is to be used only on skid trails and will only carry lightweight forestry equipment, alternate vehicle loading configurations may be used for design vehicle loads. Table 2 gives various types and sizes of forestry equipment with approximate vehicle weights and wheelbases and the results of calculations to determine the approximate maximum bending moments and shear forces for a bridge with a simple span of 9.1 m (30 ft). These design loads vary depending on the actual vehicle weight and the assumption used for load distribution. If the bridge is subjected only to these types of loads, values such as those given in Table 2 may be used for design. Since these values are all less than the value for the HS20 truck load, using such a load for design may be overly conservative.

Serviceability Criteria

Ritter (7) provided a discussion of timber bridge serviceability. Deflection in bridge members, which is one of the primary concerns in serviceability, is important for performance and aesthetics. In general, excessive deflections cause fasteners to loosen and wear surfaces, such as asphalt or concrete, to crack. Also, bridges that sag below a level plane can give the public a perception of structural inadequacy. Excessive deflections from moving vehicle loads also produce vertical movement

TABLE 2 Design Moments and Shear Forces for Various Types of Forest Harvesting Equipment Based on 9.1-m Span (AASHTO H20 and HS20 loads included)

Vehicle Type	Overall Wheelbase (m)	Approximate Loaded Total Weight (kg)	Maximum Moment for the Vehicle (kN-m)	Maximum Vertical Shear (kN)
John Deere 770B Motor Grader	6.0	15,205	214.1	112.4
John Deere 450E Crawler Tractor	2.1	7,045	139.9	61.2
Caterpillar D5H Track Skidder	2.7	19,318	367.6	160.8
Caterpillar 528 Grapple Skidder	3.3	14,091	222.3	115.8
John Deere 540D Cable Skidder	2.9	10,000	164.0	83.8
John Deere 748E Grapple Skidder	3.4	15,682	242.6	127.6
Tree Farmer C6D Forwarder	5.3	17,500	215.5	126.9
AASHTO H20-44	4.2	18,182	334.6	161.4
AASHTO HS20-44	variable	32,727	382.8	220.4

1 m = 3.3 ft 1 kg = 2.2 lb 1 kN-m = 737 lb-ft 1 kN = 225 lb

and vibration that may annoy motorists. Since most portable bridges will not need an asphalt or concrete wear surface, deflection concerns are not as great as in highway bridges.

The 1993 edition of the AASHTO bridge specifications (*6*) recommends a deflection criterion of *L*/500 for highway timber bridge superstructures. Previously recommended deflection criteria ranged from *L*/200 to *L*/1200 (*7*). Ritter (*7*) recommended maximum deflections of *L*/360 for short-term loads and *L*/240 for the combination of live and dead loads. In many portable bridge applications, it may be possible to relax these criteria as shown in Table 1.

Design, Construction, and Cost

Design

A portable timber bridge consisting of longitudinal glulam deck panels was designed and fabricated for use in a study at Auburn University to document water quality impacts from different types of stream crossing structures on temporary forest roads. This bridge was designed for AASHTO HS20 loading with relaxed deflection restrictions and is 4.9 m (16 ft) wide and 9.1 m (30 ft) log. It uses four southern pine Combination 47 glulam deck panels (*12*) 1.2 m (4 ft) wide and 267 mm (10.5 in) thick. Sketches of the bridge are shown in Figure 1. More detailed plans are available from the authors. The bridge was designed to be installed on a mud sill with the bridge deck extending 0.6 to 1.5 m (2 to 5 ft) on either side of the stream banks, and leaving an effective span of approximately 6.1 to 7.9 m (20 to 26 ft). Transverse glulam 16F-V5 stiffener beams (*12*) 171 mm wide by 140 mm deep by 4.9 m long (6.75 in by 5.5 in by 16 ft) are bolted on the lower side of the deck as shown in Figure 1. Glulam 16F-V5 (*12*) curb rails on glulam curb risers 216 mm wide by 127 mm deep (8.5 in by 5 in) are bolted to the outside deck panels. These glulam combinations are balanced layups, that is, neither side of the beam is designated as the tension or compression side, thereby reducing the possibility of installing the beam incorrectly.

The deck panels can be installed directly on the stream banks with no abutments. However, a mud sill or spread footer is recommended for placement under each end of the bridge to prevent differential settling of the deck panels into the soil. The current design specifies a southern pine Combination No. 46 (*12*) glulam mudsill that is 384 mm wide by 76 mm deep by 4.9 m long (3 in. by 15.125 in. by 16 ft). One advantage of this small sill is that less soil and aggregate material are required to build approaches to the bridge; however, larger sills may be required depending on the site conditions. All glulam components were precut and predrilled and then treated with creosote to retentions of 194 kg/m^3 (12 lb/ft^3) in accordance with American Wood Preserver's Association (AWPA) specification C14 (*10*).

The original design specified attaching galvanized ASTM A36 steel angles 203 mm by 152 mm by 12.7 mm thick by 4.3 m long (8 in. by 6 in. by 0.5 in. by 14 ft) to each end of the bridge to prevent wear on the ends of the deck panels because of vehicle traffic. These wear plates were attached with lag screws. The stiffener beams, bearing pads, and steel angles provide additional continuity to the bridge system. An additional plank or steel plate wear surface may be installed on the bridge deck depending on use conditions. Galvanized steel tie-down brackets were also provided at each of the four bridge corners to prevent bridge movement from longitudinal vehicle loads and from lateral and buoyancy forces should flooding occur. Wire rope was used to connect the steel brackets to nearby trees (deadmen could also be used to anchor the bridge). All bolts were galvanized and complied with the requirements of ASTM A307.

Construction

After the bridge components were fabricated and treated with preservative but before installation, the curb rails were attached to the deck panels to minimize the amount of erection time at the site. The bridge components were then transported to the site on a flatbed equipment trailer. Each of the panels weighs approximately 2500 kg (5,500 lb) and can be easily lifted by most knuckleboom loaders or small truck-mounted cranes. The final step in the installation process was to use a crawler tractor or skidder to level an area on each stream bank for the mud sills. The mud sills were then placed on the soil surface by using a crane or winching them into place with a skidder or crawler tractor. Then the deck panels were set in place with the same truck-mounted crane. At some sites, the soil conditions were unsuitable for the crane truck to operate safely. In these cases, a crawler tractor equipped with a winch was used to pull the panels to the stream crossing. The tractor then crossed the stream and winched the panels into place. This task could also have been accomplished by securing a snatch block on the opposite side of the stream and winching the panels across the stream. The transverse stiffener beams were then bolted in place on the bottom side of the deck panels. Although many timber bridges on low-volume roads use a plank runner wear surface, we chose not to install one on this bridge. The bridge can be installed in approximately 6 hr. In addition to a loader or crane operator, at least two people are required

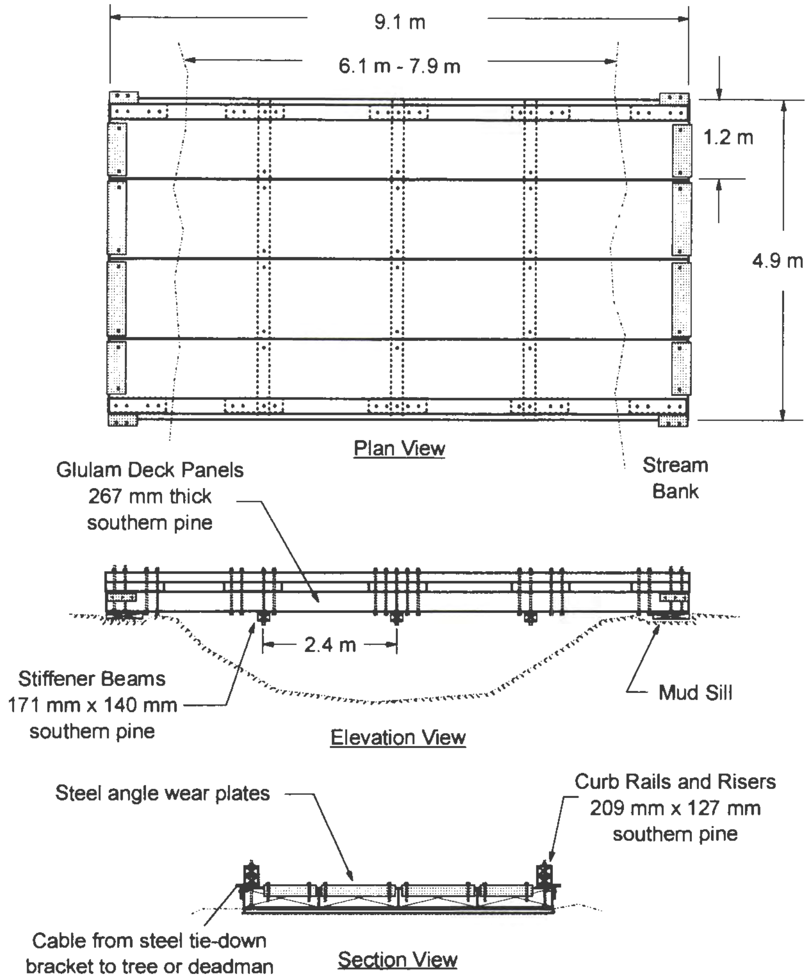


FIGURE 1 Design configuration of portable longitudinal glued-laminated timber deck bridge.

to place the components and bolt them together. The bridge can be removed in approximately 3 hr.

Cost

The total cost of the deck panels, curbs, stiffener beams, mud sills, and connectors for this bridge was $15,500 in 1993. Based on a total deck area of 44.6 m^2 (480 ft^2), the cost per square meter was $347 ($32/ft^2). A conservative estimate for the cost of one installation and removal, including transportation to and from the site, equipment operations costs, and labor to install and remove the bridge, was approximately $1,000. Distributing these costs over 10 bridge installations, the bridge would cost $2,550 per installation, which is competitve with the cost of installing a permanent corrugated metal culvert for most streams. This cost is also competitive with the commercially available steel and concrete bridges discussed previously.

Evaluation Methodology

The monitoring plans for the bridge called for stiffness testing of the individual lumber laminations, the completed glulam deck panels, and stiffener beams before bridge construction. Also, load test behavior and bridge condition were assessed. These evaluation procedures are discussed in the following sections.

Stiffness of Bridge Components

Modulus of elasticity (MOE) tests were performed at the laminating plant to determine the stiffness of each lumber specimen used in the glulam deck panels before gluing. Then, after the panels and stiffener beams were glued together, similar tests were performed to determine their respective MOE's. All of these tests were conducted using commercially available transverse vibration equipment.

Load Test Behavior

Information obtained from load tests of bridges is important in improving current design procedures for both permanent and portable timber bridges. To determine the load test behavior of this bridge, static load tests were conducted at one installation of the bridge. The tests consisted of positioning a load on the bridge deck and measuring the resulting deflections at a series of locations along the bridge centerspan and at the ends of the bridge deck panels. Measurements were taken before loading, during load application, and after the load was removed. The load used in the test was a dual-axle dump truck with a combined rear axle weight of 190.4 kN (42,800 lb). The vehicle was positioned longitudinally to the bridge so that the centroid of the rear axles was aligned with the bridge centerspan (front axles were off the bridge). Two tests were performed: one with the vehicle facing north and one with the vehicle facing south on the bridge. The vehicle was positioned transversely so that the centerline of the truck was aligned with the bridge centerline. This position resulted in an application of the wheel loads directly to the two interior deck panels; that is, the wheels did not contact either of the two outside deck panels. Measurements of bridge deflection from the unloaded to loaded condition were obtained by placing a surveying rod on the deck underside and reading values with a surveyor's level to the nearest 1.5 mm (0.06 in.). Deflection readings were taken at 8 locations across the bridge width.

Condition Assessment

The general condition of the bridge components was assessed periodically during the monitoring period. These assessments involved visual inspection of the bridge components, measurement of moisture content of the bridge components with a resistance-type moisture meter, and photographic documentation of bridge condition. Items of specific interest included the condition of the top surface of the deck panels, the curb system, the stiffener beams, the mud sill, and anchorage systems.

Results and Discussion

General Performance

The glulam bridge was installed and used at two different stream crossing sites on temporary logging roads near Auburn, Alabama, during 1993 and 1994. The bridge has performed satisfactorily under traffic loads of trucks hauling logs and chips, skidders, feller bunchers, crawler tractors, and whole-tree chippers. Although this bridge is 4.9 m (16 ft) wide, bridges used for log truck traffic might be fabricated in smaller widths. However, a narrower bridge requires additional length of straight approach roadway for proper truck tracking on the bridge.

The mud sill provided adequate support; however, the soil under the mud sills experienced as much as 152 mm (6 in.) of permanent deformation immediately after traffic began using the bridge. In sites with weak bearing capacities, a larger sill or spread footer may be necessary to prevent excessive differential settling of the deck panels.

One modification was made in the steel angle attached to the ends of the deck panels. Originally, this angle was a single piece of steel 4.3 m (14 ft) long. Although it helped provide additional stiffness and continuity to the bridge system, it was difficult to handle during installation and removal of the bridge. Therefore, before the bridge was moved the first time, the angles were removed and cut into four separate pieces. Each of these pieces was then permanently reattached to the deck panels, thereby eliminating the need to remove them during transport. Also, in case the panels needed to be skidded into place, a short loop of steel chain was welded to the vertical face of the angle.

The transverse stiffener beams provided excellent load distribution and continuity among the deck panels. The current design uses a through bolt to attach them to the deck panels as shown in Figure 1. However, this stiffener beam configuration is difficult to install in the field because it is hard to position the deck panels so that all of the bolt holes are in line. Therefore, there is a need to develop alternative panel connection methods or use other stiffener beam cofigurations like those described by Ritter (*7*) that do not use the through bolts.

Stiffness of Bridge Components

The lumber used to fabricate the glulam deck panels was nominal 50 mm by 305 mm (2 × 12) No. 1 southern pine. Results of MOE tests on this lumber before gluing indicated that it had a mean flatwise MOE of 13 652 MPa (1.98 × 10^6 psi) with a coefficient of variation of 19 percent. The design value of MOE for this grade of lumber is 11 722 MPa (1.7 × 10^6 psi) (*11*).

Tests of the four laminated deck panels resulted in a mean flatwise MOE of 13 307 MPa (1.93 × 10^6 psi). This value is in contrast to the design MOE value for Combination 47 southern pine glulam timbers, which is 9653 MPa (1.4 × 10^6 psi) (*12*). The discrepancy between the panel design MOE value and the actual MOE values may have resulted from using higher quality lumber than specified by the American Institute of Timber

Construction (*12*). Figure 2 shows the close relationship beween lumber and deck panel MOE by plotting deck panel MOE versus mean MOE of the lumber used to fabricate each respective deck panel.

Tests of the three laminated stiffener beams resulted in a mean flatwise MOE of 16 479 MPa (2.39×10^6 psi). This resulted in a mean stiffness value (MOE multiplied by the moment of inertia) of 25 270 kN-m (223,670 kip-in.), which is considerably higher than the minimum value of 9038 kN-m (80,000 kip-in.) recommended by Ritter (*7*).

Load Test Behavior

The following results are for the maximum deflections recorded under both vehicle orientations. Also, the bridge deck deflections presented account for measured deflection at the bridge supports. When subjected to the axle load of 190.4 kN (42,800 lb) placed at the bridge centerspan, the vertical deflection of the deck panels at centerspan ranged from 10.7 to 28.9 mm (0.42 to 1.14 in.). Using an effective span of 8.7 m (28.6 ft) from center of bearing to center of bearing, this maximum deflection of 28.9 mm (1.14 in.) is equivalent to *L*/300. As expected, the maximum deflection occurred under one of the interior deck panels near the centerline of the bridge, and the minimum deflection was recorded at the outer edge of one of the exterior deck panels. No permanent residual deformation was observed in the bridge deck at the conclusion of the tests.

Predicted Bridge Behavior

Design procedures listed in the AASHTO specifications for highway bridges (*6*) can be used to determine the lateral distribution of live load bending moment for longitudinal glulam timber decks. The live load bending moment for each panel is determined by applying to the panel a fraction of the wheel load (WLF), where the WLF for one traffic lane is the minimum of

$$WLF = \frac{3.28 W_p}{4.25 + \dfrac{L}{8.53}} \quad \text{or} \quad \frac{W_p}{1.68} \qquad (1)$$

where

WLF = portion of the maximum bending moment produced by one wheel line of the vehicle that is supported by one deck panel,
W_p = panel width (m), and
L = length of the span, measured center to center of the bearings (m).

The maximum deflection also can be predicted by applying the same wheel load fraction to the panel. The predicted live load deflection of each glulam panel, Δ_{LL}, is equal to the maximum deflection produced by one wheel line, Δ_{WL}, applied to a single deck panel, times the WLF. The deflection can be computed using standard methods of elastic analysis, the full moment of inertia of the deck panel, and the panel MOE adjusted for wet use conditions. Using an actual MOE of 13 307 MPa (1.93×10^6 psi), an axle load of 190.4 kN (42,800 lb), and an effective span of 8.7 m (28.6 ft), the predicted deflection at centerspan from one wheel line, Δ_{WL}, is 51.6 mm (2.03 in.). Using a panel width of 1.2 m (4 ft) and a span of 8.7 m (28.6 ft), the resulting WLF is 0.76. Therefore, when Δ_{WL} is multiplied by WLF, the predicted maximum deflection of the bridge deck, using AASHTO procedures, would be 39.1 mm

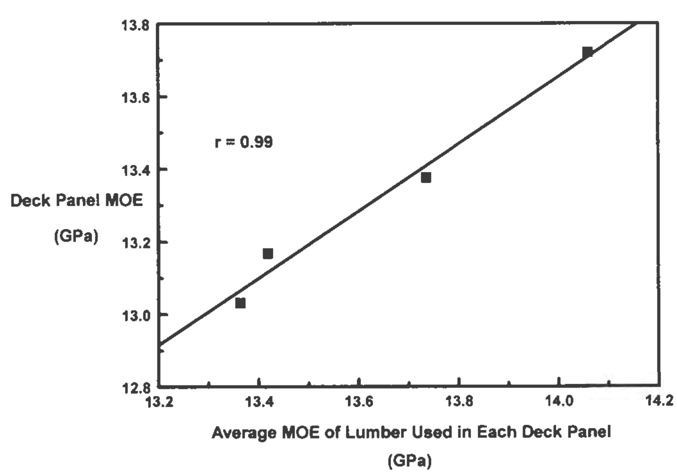


FIGURE 2 Glulam deck panel MOE versus mean MOE of lumber used to manufacture each deck panel for longitudinal glulam deck bridge.

(1.54 in.). If the recommended moisture adjustment factor of 0.833 is used, the predicted deflection now becomes 47.0 mm (1.85 in.). In both cases, the predicted deflection is considerably greater than the measured deflection of the bridge.

This discrepancy between predicted and actual deflection may have been caused by load sharing among deck panels that was greater than that predicted by AASHTO procedures. Using AASHTO procedures, the proportion of the total wheel load that should be carried by a single longitudinal deck panel is 76 percent. However, using the actual deflection data from the load tests, the equivalent proportion of the wheel load that was apparently carried by a single deck panel was approximately 56 percent.

Condition Assessment

After 12 months of use, the condition assessment of the bridge indicated that structural and serviceability performance was good. Discussion of inspection results for specific items follows.

Wood Components

Inspection of the wood components of the bridge showed no signs of deterioration. Minor checking was observed on the curb rail members and on the upper side of the deck panels because they were exposed to more wet-dry cycles. Also, very minor checking was observed in the end grain of some of the deck panels within a month after fabrication and treatment. The depths of these checks did not appear to penetrate the preservative treatment envelope of the components. Rough handling during installation and removal of the bridge resulted in minor damage to some components; however, the damage did not reduce the structural adequacy of the bridge.

Moisture content of the bridge deck panels and the stiffener beams was monitored during the use of the bridge. Although the components had a mean moisture content of 12.2 percent after fabrication at the manufacturing plant, the mean moisture content increased to 15.2 percent after 12 months. In general, the moisture content of the top surface of the deck panels was 1 to 3 percent higher than the lower surface.

There were excess creosote accumulations on the surface of the deck panels and curb rails that may have been caused by a creosote retention higher than that specified [assay results from the treatment plant showed an actual retention of 292 kg/m^3 (18 lb/ft^3) in the deck panels instead of the 194 kg/m^3 (12 lb/ft^3) specified]. Other preservatives, such as chromated-copper arsenate (CCA) or pentachlorophenol, may be more desirable for a bridge that will be handled several times. The primary advantage of creosote over the other treatments is its ability to form an envelope that prevents the wood from absorbing water.

Bridge Deck Surface

The steel angle wear plates installed on the ends of the deck panels appeared to be preventing damage from traffic as it drove onto the bridge deck. But, since the bridge deck was constructed without an additional wear surface, particular emphasis was placed on observing damage to the top surface of the deck panels. Gravel and other debris carried onto the bridge by traffic have left numerous gouges in the surfaces of the deck panels; however, none of these gouges are deep enough to reduce the structural adequacy of the bridge and none of them have penetrated the preservative treatment envelope. Therefore, an additional wear surface does not appear to be needed.

Summary and Conclusions

Cost-effective portable bridge designs are needed for temporary low-volume roads. Although there is much new technology in timber bridges, little research has applied this technology to portable bridge systems. Many of the advantages of timber bridges, which are lightweight and easy to fabricate and install and which can be prefabricated, make them ideal for temporary stream crossings on low-volume roads.

Previous research on glulam military bridges and on bridges for low-volume roads indicates that, although using the AASHTO design procedures for portable timber bridges is safe and conservative, it may result in overly conservative and uneconomical designs. A matrix of proposed design criteria presented here suggests that many of the AASHTO criteria can be modified for portable bridge systems. However, additional research is needed on design procedures and strength and serviceability criteria for portable timber bridges.

The design of a portable longitudinal glulam timber deck bridge was presented in this paper. Based on tests of this bridge over a period of 12 months, the following specific conclusions are given:

1. It is economically feasible to fabricate and use portable timber bridges for temporary stream crossings on low-volume roads.
2. Portable timber bridges can be successfully installed, used, and removed without sustaining substantive damage.

3. Load testing and analysis indicate that the longitudinal glulam bridge system is stiffer than AASHTO design procedures predict. The predicted deflection at the bridge midspan was 47.0 mm (1.85 in.), and the actual deflection was 28.9 mm (1.14 in.), or *L*/300, under an axle load of 190.4 kN (42,800 lb).

4. Greater distribution of vehicle loads than predicted by AASHTO appears to be occurring among the deck panels in the longitudinal glulam deck system. When load test data were used, the apparent WLF was 0.56; when AASHTO procedures were used, the WLF was 0.76.

5. The bridge deck panels have withstood abuse from vehicle traffic without needing an additional wear surface.

ACKNOWLEDGMENT

Approved as Alabama Agricultural Experiment Station Journal No. 2-944734. This research was supported by the USDA Forest Service Forest Products Laboratory and by the Alabama Agricultural Experiment Station.

REFERENCES

1. Rothwell, R. L. Erosion and Sediment Control at Road-Stream Crossings. *Forestry Chronicle*, Vol. 23, 1983, pp. 62–66.
2. Swift, L. W. Forest Road Design to Minimize Erosion in the Southern Appalachians. In *Proceedings, Forestry and Water Quality: A Mid-South Symposium* (B.G. Blackmon, ed.), Cooperative Extension Service, University of Arkansas, 1985, pp. 141–151.
3. Alt, B. C. *Portable Pre-Stressed Concrete Logging Bridge*. American Pulpwood Association Technical Release 91-R-64. American Pulpwood Assoc., Washington, D.C., 1991.
4. Hassler, C. C., M. P. Wolcott, B. Dickson, R. E. Driscole, and W.B. Perry. A Modular Timber Bridge for Temporary Stream Crossings. In *Proceedings of the 13th Annual Meeting of the Council on Forest Engineering* (A. E. Hassan and E. L. Deal, eds.), North Carolina State University, 1990, pp. 190–194.
5. Taylor, S. E., and G. L. Murphy. *Portable Timber Bridge Designs for Temporary Forest Roads*. ASAE Technical Paper 92-4559. ASAE, St. Joseph, Mich., 1992.
6. *Interim Standard Specifications for Highway Bridges*. AASHTO, Washington, D.C., 1993.
7. Ritter, M. A. *Timber Bridges: Design, Construction, Inspection, and Maintenance*. USDA Forest Service, Washington, D.C., 1990.
8. Knab, L. I., R. C. Moody, W. W. Sanders, and H. A. Elleby. *Design Criteria for Theater of Operations Glued-Laminated Timber Highway Bridges*, Vol. I. Technical Report No. M-198. U.S. Army Engineer Construction Engineering Research Laboratory, Champaign, Ill., 1977.
9. GangaRao, H. V. S., and T. R. Zelina. Development of Low-Volume Road Bridges. *Journal of Structural Engineering*, ASCE, Vol. 114, No. 9, 1988, pp. 1941–1961.
10. *Standards*. American Wood Preserver's Association, Woodstock, Md., 1991.
11. *Design Values for Wood Construction: A Supplement to the 1991 Edition of the NDS*. American Forest and Paper Association, Washington, D.C., 1991.
12. *AITC 117-93 Design, Standard Specifications for Structural Glued-Laminated Timber of Softwood Species*. American Institute of Timber Construction, Englewood, Colo., 1993.

Low-Level Stream Crossings in Developing Areas

Eddie Otte, *Van Wyk & Louw Inc., South Africa*
Andries J. Pienaar, *Northern Transvaal Province Department of Public Works, South Africa*

Lack of funds has delayed the provision of stream crossings in the developing rural areas of South Africa. A fund to promote labor-intensive construction and the realization that design standards could be reduced to allow for a 1-in-2-year flood flowing 300 mm (1 ft) deep across the road provided an opportunity to meet some of the community needs. The Roads Department accepted that floods bigger than a 1-in-2-year flood would overtop the structures and cause damage requiring maintenance. Designs were simplified to ensure labor-intensive construction. Three typical designs were used: a concrete slab or causeway so that the water would flow over the road, a concrete slab with one or more openings to accommodate the base flow while floods flow over the road, and a bridge or culvert with several openings, usually with a simply supported concrete deck. Liberal use was made of gabions and cut-off walls in all three designs to reduce erosion damage at the inlets and outlets. After calling for bids, the Roads Department awarded five separate contracts during 1992 to construct 17 low-level stream crossings. Special conditions in the contract documentation were necessary to ensure labor-intensive construction. The use of labor-intensive construction techniques for low-level crossings designed to reduced standards was very successful. These stream crossings met both technical and social needs in a developing area. The program will be continued and additional funds will be provided.

In the relatively arid developing rural areas of the northern Transvaal in South Africa, stream crossings on gravel roads carrying less than 50 to 100 vehicles per day generally had very vew drainage structures. As these streams have a relatively limited flow, are usually dry for fairly long periods during the year, and have relatively low traffic volumes, the public in general accepted the situation. However, during the rainy season, vehicles often got stuck in the natural streambed. The Roads Department was aware of the problem, but because funds were very limited, other projects usually took precedence. When the central government made funds available for job creation and the promotion of labor-intensive construction methods in the rural areas, the opportunity was seized (*1*).

This paper does not report on original research findings. It is a practical paper about reduced design requirements for drainage structures to provide an engineering solution using labor-intensive construction methods. All the work had to be executed within the constraints of limited available funds and such local conditions as general lack of skilled labor.

Design Standards

The flood recurrence interval for the design of drainage structures usually depends on the level of service, traffic volume on the road, and the magnitude of the expected flood (2). For example, a bridge over a river on a major road may be designed to accommodate a 1-in-50-year flood, whereas a culvert [for example, with one or two 600-mm (2-ft) diameter stormwater pipes] across a small stream on a minor road may be designed to accommodate only a 1-in-5-year flood. Bigger floods would overtop the road and probably cause erosion damage that would require maintenance.

Prevailing conditions (needs, funds, traffic) in the area demanded reduced construction costs. It was subsequently decided to reduce the design standards by accepting that the flow depth of the 1-in-2-year flood would be less than 300 mm (1 ft) and that bigger floods would overtop the structure for a maximum period of 24 hr (*1*). A corollary was reduced construction costs but increased maintenance costs, for example, for repairing the approaches after severe storms. Maintenance costs and the waiting cost for the traveling public because of flooding were included in the economic analysis to establish a priority rating for the different structures.

A substantial saving is possible if crossings are constructed for one-lane traffic only, but the prevailing traffic conditions (buses, speed, and dust) required designs for two-lane traffic. The clearance width among the guide blocks on the edges of the structure was generally 8 m (26 ft 3 in.). To reduce costs, no guardrails were provided, and guide blocks 300 mm (1 ft) high were provided at 4-m (13-ft) intervals. While the guide blocks are still visible, that is, when the water depth is less than 300 mm, vehicles would be able to negotiate the crossing safely. A disadvantage of guide blocks is the adverse effect on the self-cleaning properties of the structure after a flood with subsequent silt deposits on the road. This disadvantage had to be accepted because guide blocks are a necessary safety feature.

The traffice loading used in the design of the bridge decks was also adjusted downward. The normal loading code of the Roads Department was applied, but the abnormal heavy vehicle [i.e., 36 kN/m^2 (5.2 $lbf/in.^2$)] was omitted. The decks were designed to withstand a uniformly distributed load of 10.5 kN/m^2 (1.52 $lbf/in.^2$) with a single knife-edge load of 39 kN/m (2,670 lbf/ft) and a 16-wheel heavy vehicle with four axle loadings of 320 kN (72,000 lb) each, that is, 80 kN (18,000 lb) per wheel.

Labor-Intensive Construction

One objective of labor-intensive construction is to provide job opportunities for the unemployed. This objective can be achieved through manual labor but with due regard to technical quality and efficiency of construction (*3*). The type and size of the low-level structures decided upon were ideally suited to labor-intensive construction. The fact that the financier wanted to create job opportunities and promote labor-intensive construction was additional motivation for adopting this construction method.

Typical Structures

For the purposes of labor-intensive construction and in order to accommodate the experience and ability of the labor force, the structure design had to be simplified. Based on both the hydraulic (e.g., design flood) and the geometric (e.g., vertical curve) requirements, three typical designs were accepted:

1. A concrete slab or causeway in which all the water would flow over the road. The length of the concrete slab and the vertical curvature (i.e., the *k*-value) were selected to ensure a flow depth of less than 300 mm (1 ft) during the 1-in-2-year flood (Figure 1).
2. A concrete slab with one or more openings at the bottom to accommodate the base flow and to increase the hydraulic capacity to ensure compliance with the design standards (Figure 2).
3. A bridge or culvert with several openings to accommodate an extensive flood. These structures were generally built with stone masonry walls and a simply supported cast in situ concrete deck (Figure 3).

In all instances, potential flood damage had to be minimized. Liberal use was therefore made of gabions for protection. Gabions 3 m (10 ft) and 5 m (16 ft 6 in.) wide were provided upstream and downstream of virtually all the structures. If the structure was built on rock or special conditions prevailed, the gabions were omitted. Cut-off walls were constructed at all the structures, both upstream and downstream, down to bedrock or to at least 800 mm (2 ft 6 in.) below the streambed. These precautions were taken to minimize scour damage.

These designs were deemed labor-intensive because the major components were excavating, building stone masonry walls, mixing and placing concrete, and packing gabions. The bids and the outcome of the project actually confirmed this assumption. None of the bidders decided to hand-mix the concrete; they all elected to use relatively small mechanical concrete mixers.

Contract Requirements

This particular program was one of the first for which bids were obtained to perform labor-intensive construc-

FIGURE 1 Concrete slab constructed so that water could flow over structure.

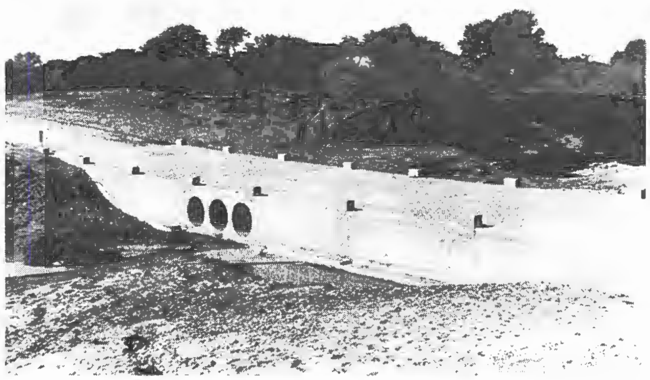


FIGURE 2 Concrete slab with three openings.

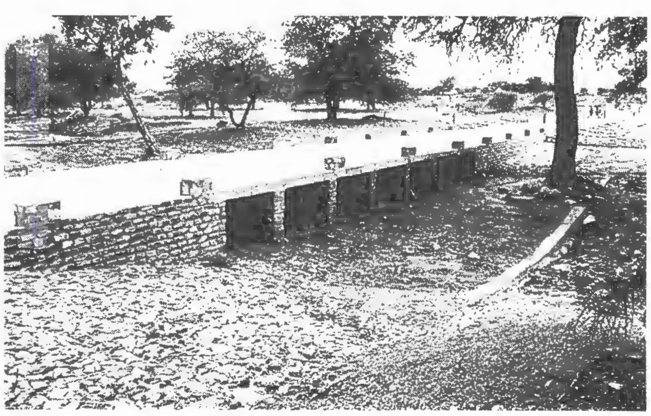


FIGURE 3 Stone masonry bridge (note guide blocks on deck and gabions and cut-off wall in foreground).

tion. This program presented the construction industry with a new requirement—a number of job opportunities had to be created during the execution of the contract.

The contract documentation was compiled to ensure labor-intensive construction, and bidders had to both bid an amount of money and undertake to create a certain number of job opportunities. Both aspects were considered in awarding the contract.

The bidder had to plan properly when bidding because detailed information had to be provided on the number of laborers who would be employed. The supervising engineer had to check on the numbers regularly, and, if fewer laborers than those bid were on site, a substantial penalty was assessed. This penalty was considered necessary to ensure that the specified number of job opportunities was provided. There were no difficulties with this "new" requirement, and it was not necessary to enforce the penalty.

Program Details

On the basis of the available funds, a cost-benefit study, and the needs of the area, 17 stream crossings were constructed in early 1992 as five separate contracts. The bid amounts varied from $100,000 to $300,000, and on average the cost of a job opportunity (i.e., labor cost, provision of materials, and professional fees) varied between $330 and $500 per month.

Discussion and Recommendation

Reduced design standards and labor-intensive construction availed the Roads Department of an opportunity to provide stream crossings in developing areas where it would otherwise not have been possible. It is accepted that these crossings are not perfect in all respects, but they are considered adequate.

The traveling public accepted the structures enthusiastically. Due to the success of this program, the Roads Department decided to continue with the provision of low-level crossings; more funds will be made available to construct crossings based on the reduced standards.

The combination of reduced design standards and labor-intensive construction presented an economic solution to an urgent technical and social problem. It is recommended that this program be implemented elsewhere in the developing world where similar circumstances prevail.

ACKNOWLEDGMENT

The permission of the Department of Public Works to publish this technical note is acknowledged.

REFERENCES

1. Otte, E., A. J. Pienaar, C. Briers, and E. Wood. *Labour-Intensive Bridge Construction in Lebowa: A Case Study*. Presented at the Symposium on Labour-Intensive Construction, Eskom Training College, Midrand, South Africa, March 25–26, 1993.
2. National Transport Commission. *Road Drainage Manual*. Directorate Land Transport, Department of Transport, Pretoria, South Africa, 1983.
3. Development Bank of Southern Africa. *Interim Guidelines for Labour-Based Construction Projects*. Construction and Development Series, Number 2. Development Bank of Southern Africa, Halfway House, South Africa, Feb. 1993.

Watershed Management with Respect to Low-Volume Road Drainage Detention Structures

Thomas L. Behm, *Minnesota Department of Transportation*

The availability of state and federal monies to replace local bridges has allowed engineers to evaluate watershed management in conjunction with the bridges' physical replacement. By using computer-driven watershed models, engineers can incorporate flood reduction measures into bridge replacements. Bridge and culvert openings have been restricted and modified to temporarily store flood runoff on low-volume county and township roads. The result can be an economical bridge replacement with a significant reduction in peak runoff. Properly designed and constructed, road detention structures can modernize the rural transportation network of farm-to-market roads. The background and procedure for designing detention structures are outlined.

Over the past several years, Lyon County, Minnesota, has begun using a series of creative flood control practices on local roads to reduce damage to roads and bridges. In various situations and in different combinations, Lyon County has restricted culverts and bridge waterway openings and created temporary flood storage to meet the overall goal of regional watershed management. To date, nearly 40 projects have been constructed in Lyon County, saving project construction costs and significantly reducing peak runoff discharge.

Background

Flood damage reduction is a high priority for Lyon County, which averages about $26,000 in damage to roads and bridges each year. Damages to local roads and bridges occurred frequently in 1957, 1969, 1983, and 1993. Damage for a single event has been as high as $830,000 for county and township structures. Soil loss, streambank, and crop loss estimates by the Soil Conservation Service are as high as $500,000 per year. In 1989 the county received support from the Department of Natural Resources (DNR) to incorporate flood storage into county and township bridge replacement projects. A two-staged outlet structure sized to the reservoir has reduced the peak runoff by as much as 80 percent with less cost than a conventional design.

Typically, bridges and culvert structures are designed to pass the 100-year or 50-year runoff events with little stage increase. The resulting hydrographs therefore show very little difference between inflow and outflow and little runoff storage. The state and federal bridge replacement programs have provided the impetus for this type of design by minimizing the financial commitment from the local unit of government.

Since the program's initiation, Lyon County has evaluated 80 potential road and bridge projects based on terrain, land use, hydraulic, and economic considerations. For each project, the drainage area, the flood pool size, volume of flood storage available, the percent reduction in peak discharge for the 10-year and 100-year floods, and projects costs were identified. From this information, several ratios to help prioritize potential storage sites were developed. These ratios included volume of flood storage to area, cost per acre-foot storage, and cost relative to reduction in peak runoff. Typical projects have an average cost per structure of less than $100,000, which is economical for a multipurpose project compared to costs for bridge replacement, flood damage, and crop loss (typically 25 to 35 percent higher).

Example Projects

The first project completed was on Three Mile Creek, a tributary to the Redwood River. This project involved replacing an existing small bridge on a county road that has routinely washed out. The land upstream of the crossing is pastured and therefore not damaged by short-term flooding. For this project, Lyon County put in a 12-ft-wide by 7-ft-high box culvert with a V-notch weir constructed into the upstream apron (Figure 1). The road was raised a maximum of 6 ft for a distance of 1,000 ft.

Because of the large watershed (14.0 mi^2), the county needed additional storage areas. The DNR agreed to the temporary flooding of the property to the northeast of this crossing, part of the Furgamme Wildlife Management Area (WMA). In conjunction with the V-notch weir, a 48-in.-diameter culvert was installed into the Furgamme WMA so that, during flooding, water could back up into Furgamme WMA and flow back out as water recedes on Three Mile Creek.

Another complication was that the old crossing had a 6-ft overfall that the DNR wanted maintained because it served as a fish barrier for rough fish migrating to Goose Lake, a walleye-stocked and aerated lake, located 2 mi upstream. A cutoff wall was constructed to support the end of the apron along with rock gabions on the outlet of the box culvert. The gabions were necessary because of the steepness of the road ditch following the raising of the road and high outlet velocities expected through the culvert.

The total project cost was $75,230, of which approximately $30,000 was associated with the raised road, the diversion culvert, and the specially constructed V-notch weir apron. The project reduced the 100-year peak discharge by 40 percent. The upstream landowner in this case was given a one-time easement payment of $200 per acre for the 85 acres inundated by the 100-year flood.

The second road project completed was on the Cottonwood River, a major tributary to the Minnesota River. The drainage area on this project was 26.0 mi^2 of agricultural land. In this case, Lyon County decided to replace the existing bridge that had 196 ft^2 of waterway area with a 12-ft-wide by 10-ft-high box culvert. The inlet of the box culvert was modified with a 36-in.-diameter, low-flow culvert designed to allow the normal flow of water through the structure (Figure 2). When the capacity of the 36-in. culvert was exceeded, the structure would impound approximately 8.5 ft of water over an 80-acre pool. As runoff increased, the flow then started over the drop box inlet and through the 12-ft by 10-ft box culvert; in this case the road was raised a maximum of 6.2 ft for a distance of 550 ft.

The upstream property was again pastured seasonally. Because the county could not acquire enough upstream land rights for an extremely effective flood control project, they decided to try to develop this area for other uses. Since the county owned the land immediately upstream of the structure and topsoil was needed

FIGURE 1 V-notch weir.

FIGURE 2 Cottonwood River box inlet structure.

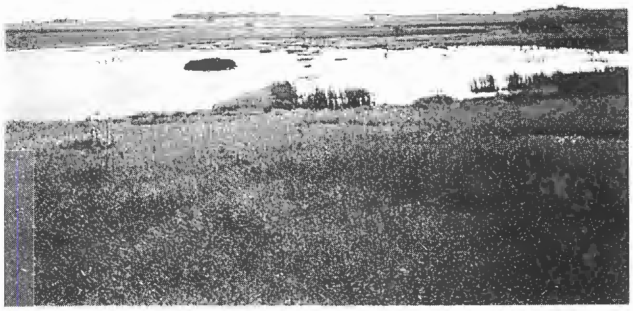


FIGURE 3 Cottonwood River wetlands.

for road projects and the county landfill cover, it was decided to go into the area above the structure and excavate topsoil. Lyon County excavated about 3 ft of material and left five nesting islands for waterfowl within a shallow permanent pool wetland area (Figure 3). This multipurpose project has become a model for the county DNR. The cost was $45,000 for the structure and excavation of the roadway, and the peak flow as reduced by 20 percent.

Recommendations

The other 38 projects are of similar design. Most have fairly small watersheds and involve installing a low-flow pipe with a concrete riser of different heights and sizes depending on the storage available and watershed size. On most projects, the road is raised substantially; for example, one road was raised 25 ft. In addition to decreasing the time that some roads are inundated by floodwaters, the road fill serves several other substantial purposes, including the improvement of sight distance, wetland criteria, wider structure for farm equipment, and less winter maintenance.

Inlet types have generally been drop inlet spillways using precast concrete elements. Flow characteristics of the drop inlet will vary according to the proportional sizes of the different elements. Typically, the free-falling overflow drops vertically into the base of the structure; a plunge pool can be induced by placing impact blocks at the base of the structure to help dissipate energy. The purpose is to dissipate energy within the drop structure and not subject the outlet to excessive stream velocities.

Consideration must be given to the high heads and resulting increased outlet velocities and the dissipation of energy and erosion control downstream. Energy dissipating rings, drop inlets, and hydraulic jump stilling basins have been used with consistent success.

Commercially available software similar to that of the Soil Conservation Service (SCS) TR-20 and TR-55 programs has been used. The process of analyzing runoff, routing, ponds, and hydraulics is extremely quick and readily allows investigation of alternative designs.

Design Procedure

The following is an example of the reports and graphics for a small project involving a single watershed and structure.

This project has a watershed area of 1,772 acres with a weighted curve number of 78, which is applied to a SCS Type II, 24 hr rainfall of 5.7 in. The structure in this case is on a county highway and replaces a 25-ft span bridge. The selected structure is a low-flow 36-in.-diameter culvert inlet at the natural flow line. This inlet connects to a 60-in.-diameter outflow pipe with a 60-in.-diameter vertical orifice at a point 29 ft above the flow line (Figure 4).

The runoff hydrograph calculations are made based on the watershed characteristics (Figure 5). The resulting stage versus discharge curve is shown in Figure 6.

The stage-storage data are calculated from the pond surface area by the prismatic method. The result is the inflow/outflow hydrograph indicating the peak elevation, peak storage, and drawdown time (Figure 7).

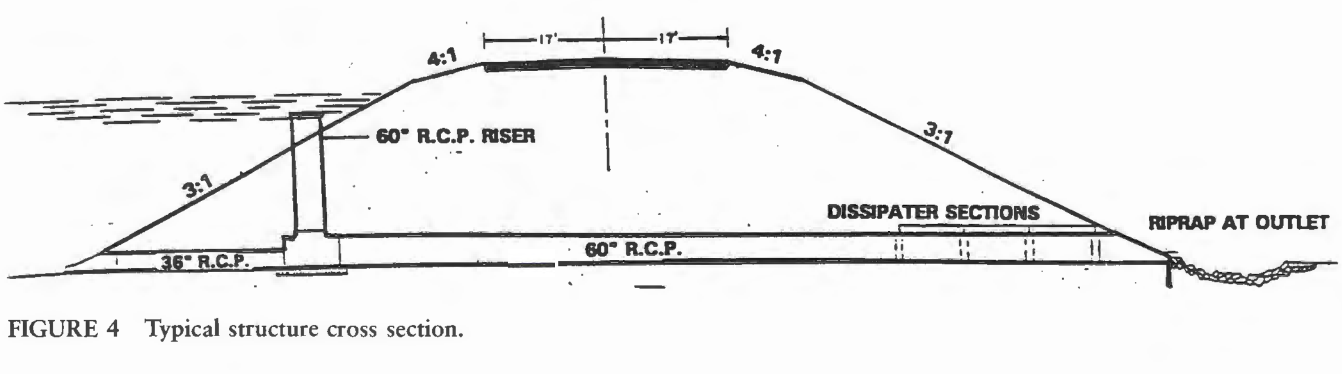


FIGURE 4 Typical structure cross section.

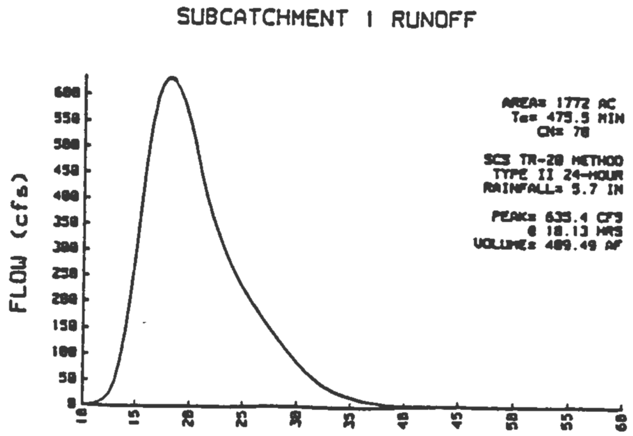


FIGURE 5 Runoff hydrograph.

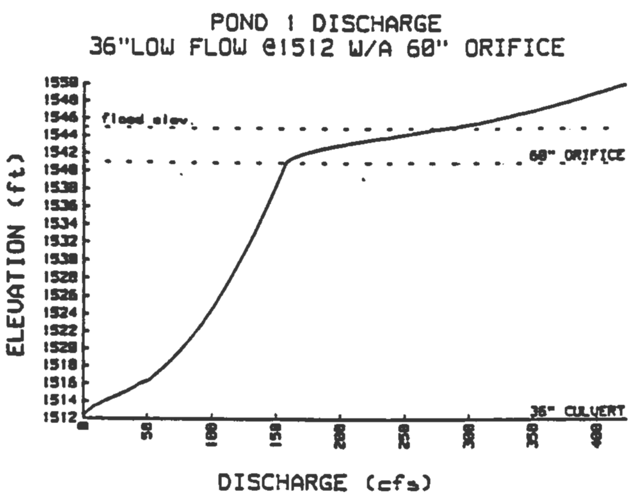


FIGURE 6 Stage versus flow graph.

In the case of this 100-year rainfall event, the peak discharge was reduced by 55 percent with a cost savings in the bridge versus retention structure of $60,000. Most projects constructed provide similar cost savings and flood protection.

Conclusion

The incorporation of flood storage into road and bridge projects is well accepted by the public and local officials. The road structure dam is designed with its top wide enough to accommodate a road and has an outlet that slowly drains away stormwater impounded above

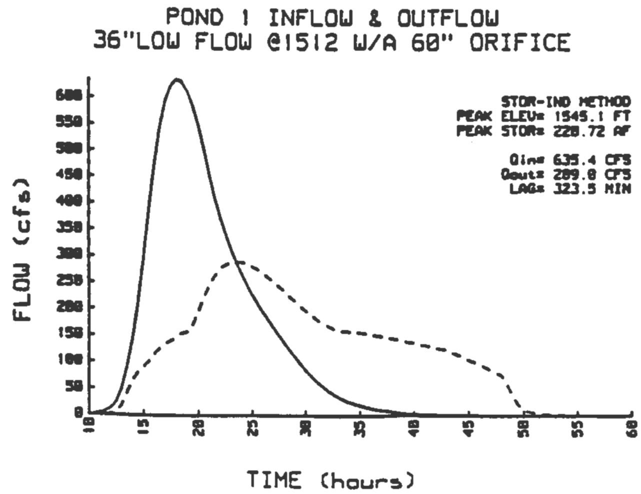


FIGURE 7 Inflow-outflow hydrograph.

the permanent pool. These projects are subject to the Minnesota Department of Transportation's requirements for standards and funding and DNR environmental concerns.

Lyon County has an annual budget of $300,000 per year for these types of projects and a goal of 80 projects by the year 2000. In numerous cases, the dam on the road can have more beneficial results than a replacement bridge or even a large pipe. The road structure will cost one-fourth to one-third that of a new bridge and will reduce the peak runoff to one-fourth to one-third of the peak discharge.

A road detention structure is usually a multipurpose project, because it can provide

1. A roadway across the top of the dam;
2. Flood control by slowly releasing runoff from intense storms;
3. Erosion control by stabilizing the stream grade;
4. Improved downstream water quality by trapping sediment;
5. A potential site for recreation;
6. Wildlife habitat areas, including the reservoir itself and planting sites upstream, downstream, and adjacent to the reservoir;
7. A possible source of water for irrigation or other farm needs, and
8. An economical bridge replacement.

Long-range plans call for developing similar projects in adjacent counties that affect Lyon County's tributaries.

Field Performance of Stress-Laminated Timber Bridges on Low-Volume Roads

Michael A. Ritter and James P. Wacker, *Forest Products Laboratory, USDA Forest Service*
Sheila Rimal Duwadi, *Federal Highway Administration*

Stress-laminated timber bridges were first introduced in the United States in the late 1980s. Since that time, the concept of stress-laminating has received a great deal of attention and hundreds of bridges have been built. Most of these bridges are located on rural low-volume roads. To evaluate the performance of stress-laminated bridges, the United States Department of Agriculture (USDA) Forest Service, Forest Products Laboratory, implemented a nationwide bridge monitoring program in 1988 that was subsequently expanded in 1992 to include a cooperative program with the Federal Highway Administration. This paper presents a summary of monitoring results and observations obtained through that program for stress-laminated bridges that have been continuously monitored for 2 years or more. Included are discussions related to bridge construction, moisture content, stressing-bar force, vertical creep, load test behavior, and condition evaluation. Based on the monitoring program results, performance of stress-laminated timber bridges is generally satisfactory, although observed performance can be improved in several areas.

Stress-laminated timber bridge decks consist of a series of wood laminations that are placed edgewise between supports and stressed together with high-strength steel bars (Figure 1). The bar force, which typically ranges from 111 to 356 kN (25,000 to 80,000 lb), squeezes the laminations together so that the stressed deck acts as a solid wood plate. The concept of stress-laminating was originally developed in Ontario, Canada, in 1976 as a means of rehabilitating existing nail-laminated lumber decks that delaminated because of cyclic loading and wood moisture content variations (*1,2*). In the 1980s the concept was adapted for the construction of new bridges, and numerous structures in Ontario were successfully built or rehabilitated using the stress-laminating concept. The first stress-laminated bridges in the United States were built in the late 1980s. Since that time, several hundred stress-laminated timber bridges have been constructed, primarily on low-volume roads. Although most stress-laminated bridges are slab-type bridge decks constructed of sawn lumber or glued laminated timber (glulam), the technology has also been extended to systems employing stress-laminated trusses and T-beam and box sections. The scope of this paper is limited to slab-type deck applications.

Stress-laminated timber decks are characterized by several features that make them particularly attractive for low-volume roads where relatively short single spans up to approximately 11 m (36 ft) are required. From a materials aspect, stress-laminated bridges generally require smaller-sized, lower-quality lumber than

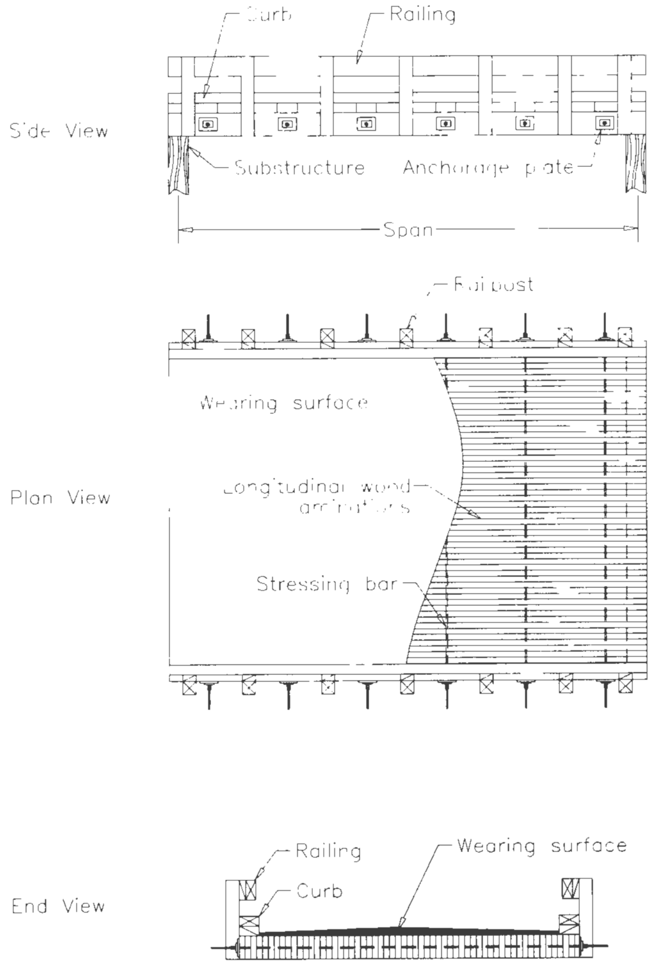


FIGURE 1 Typical configuration of stress-laminated timber deck bridge.

is typically required for other types of mechanically laminated timber decks. Because load transfer between the deck laminations is developed by friction, all laminations need not be continuous over the bridge span, and butt joints are permitted within certain limitations (Figure 2). This reduces the length of lumber required and is more conducive to the use of locally available wood species. In addition, the laminating process disperses natural defects in the wood so that variability is reduced and higher design values are possible. The bridges are also relatively simple to build and are often assembled by local crews in one day or less.

Design procedures for stress-laminated decks were first included in the Ontario Highway Bridge Design Code (OHBDC) in 1983 (*3*). In the United States, research on stress-laminated bridges in the late 1980s led to the development and publication of several design methods (*4,5*). In 1991, stress-laminated timber decks were recognized in a national design specification published by AASHTO (*6*). Since 1991, most stress-laminated timber decks have been designed in accordance with AASHTO specifications, which recommend a maximum live load deflection equal to 1/500 of the bridge span. When this approach is used, the design is normally controlled by serviceability requirements for stiffness rather than strength requirements.

To evaluate the field performance of stress-laminated bridges, the Forest Products Laboratory (FPL) of the U.S. Department of Agriculture (USDA) Forest Service, implemented a nationwide bridge monitoring program in 1988. In 1992, this program was expanded through a cooperative program with the Federal Highway Administration (FHWA). The purpose of the program is to monitor and evaluate bridge performance and behavior in order to develop, confirm, or improve methods of design, fabrication, and construction by obtaining representative information on the performance of different bridge designs and materials under various geographical and environmental conditions. This paper presents monitoring results for stress-laminated bridges that have been continuously monitored for 2 years or more. Included are observations and discussions related to bridge construction, moisture content, stressing-bar force, thermal response, vertical creep, load test behavior, and condition evaluation.

Bridge Monitoring

Bridges included in the bridge monitoring program are selected on the basis of location, configuration, wood species, and preservative treatment. In most cases, the monitoring is undertaken as a cooperative research venture with the bridge owner. Local personnel play a key role in collecting field data. Data on each bridge are normally collected over a period of 2 to 3 years and involve monitoring methods developed by FPL (*7*). Key monitoring activities and methods are summarized below.

- Bridge construction: Information on bridge construction is obtained by visiting the bridge site and documenting the construction sequence and methodology. This task is commonly completed by the monitoring cooperator in the form of written and photographic documentation of the construction process.
- Moisture content: The moisture content of the bridge deck is typically measured with a electrical resistance-type moisture meter at 6 to 12 locations. Moisture content readings are generally taken on a monthly or semiannual basis depending on various bridge parameters. In addition, core samples may be removed from the deck to determine moisture content in the laboratory and calibrate electrical resistance readings more accurately. Core samples may also be taken

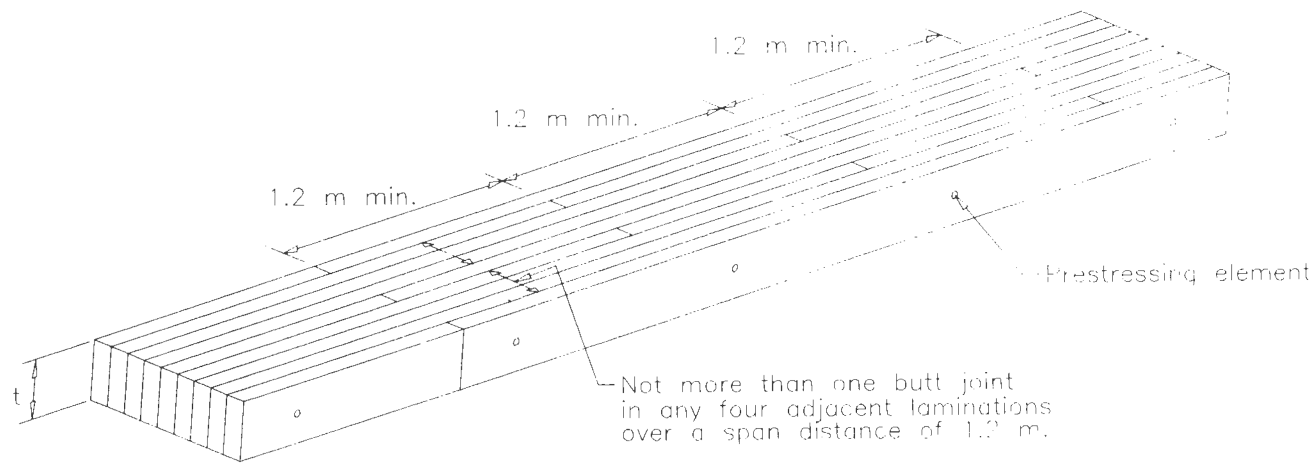


FIGURE 2 Common butt joint configuration used in stress-laminated timber bridges.

when the accuracy of the electrical resistance meter is questionable, such as when waterborne preservatives are used or when the deck moisture content exceeds the fiber saturation point (approximately 30 percent).

- Stressing-bar force: To monitor stressing-bar force, two to three load cells are installed on each bridge. The strain in the load cell is measured manually with a portable strain indicator or automatically through a remote data acquisition system. The measured strain is then converted to an equivalent stressing-bar force. Manual readings are normally taken on a monthly basis but may be as frequent as weekly for a period immediately after construction. With a remote data acquisition system, readings are automatically recorded several times a day.
- Thermal response: The response of stress-laminated decks to temperature changes is measured with thermocouples installed at various locations in the bridge deck. Deck temperatures are then compared with ambient temperatures and load cell readings to evaluate bridge response to temperature change. Thermal response requires the use of a remote data acquisition system, and readings are recorded automatically at the same frequency as load cell readings.
- Vertical creep: Long-term vertical creep is measured by referenced elevation readings taken on the bridge underside at centerspan. Such readings are typically obtained with a survey rod and level or with calibrated rules that are suspended from the bridge underside and read relative to a reference stringline.
- Load test behavior: Bridge behavior under vehicle loading is determined for various vehicle positions by measuring the relative displacements of the bridge deck from an unloaded to loaded condition. For single-lane bridges, one vehicle is used. For two-lane bridges, one vehicle is used in each lane, and both lanes are loaded simultaneously. Deflection measurements are obtained by suspending calibrated rules from the deck underside and reading the relative position of the rules with a surveyor's level or by measuring the bridge deflection directly with displacement transducers.
- Condition evaluation: A condition evaluation of each bridge is completed several times during the monitoring period and involves intensive visual inspections and photographic documentation. Specific evaluation areas include the general structure condition, stressing system corrosion, and wearing surface performance.

The FPL/FHWA bridge monitoring program currently includes approximately 40 stress-laminated timber bridges located across the United States. Each year, five to eight new bridges are added, and approximately the same number of bridges are completed. The information presented in this paper is based on the data obtained over the past 6 years from 24 bridges that have been continuously monitored for periods of 2 years or more. Performance trends and conclusions are representative of the general behavior demonstrated by the bridges. More specific information on individual bridges will be available in the future as more information is obtained and detailed reports are published. Additional information on stress-laminated bridge performance is also available in reports published by Wacker (*8*), Dickson and GangaRao (*9*), Gutkowski and Lewis (*10*), and Mozingo and DiCarlantino (*11*).

Field Performance of Stress-Laminated Decks

The field performance of stress-laminated timber deck bridges has generally been satisfactory. When proven design and construction practices are followed, performance has typically been good. However, deviations from recommended practice have resulted in unexpected performance problems in some cases. The majority of these problems have been related to serviceability rather than the structural (safety) features and have resulted from the evolutionary nature of the stress-laminated system in the United States. Although proven design and construction criteria have been developed in Ontario for a number of years, definitive guidelines on design, construction, and maintenance practices have taken time to evolve in the United States. In addition, many U.S. designs have differed from those in Ontario and do not necessarily fit within existing standards of practice. One method to improve field performance of stress-laminated timber bridges is to learn from past experience and incorporate proven technology into future bridges.

The following is a summary of the field performance information related to stress-laminated timber bridges included in the FPL/FHWA bridge monitoring program. Included are information and observations related to bridge construction, moisture content, stressing-bar force, vertical creep, load test behavior, and condition evaluation.

Bridge Construction

A number of methods have been used to construct stress-laminated timber bridges (*4*). When laminations are continuous (i.e., no butt joints), they can be individually placed on abutments, bars can be inserted, and the bridge stressed in place. When butt joints are used, the bridge can be prefabricated into nailed or banded panels that are stressed together in the same manner.

In addition, bridges may be prefabricated into prestressed panels that are joined with bar couplers at the bridge site. Regardless of the construction method used, current practice requires that stress-laminated timber bridges be stressed three times during the construction process: at initial assembly, 1 to 2 weeks after the first stressing, and 4 to 6 weeks after the second stressing (*4*). Most bridges in the United States have been stressed using one jack rather than the multiple jacks commonly used in Ontario. This issue is primarily economic because the high cost of a multiple-jack system cannot be justified unless a large number of stress-laminated bridges are built on a continuing basis. A single-jack system can be purchased for approximately $1,200 and provides similar results if proper bridge stressing procedures are followed.

Field monitoring has shown that construction methodology and practices can affect bridge performance and appearance. When using a single jack for stressing, the most frequent problems result from a failure to recognize that, as the bars are stressed during construction, the laminations are compressed and the bridge width narrows. This narrowing is generally most pronounced during the first stressing but may also occur during the second stressing at a decreased level. By the third stressing, deck narrowing is minimal. The magnitude of deck compression during the first stressing can vary from 25 to 75 mm (1 to 3 in.) depending on a number of factors including the bridge width, wood species, and the straightness of the lumber laminations. More compression occurs as the bridge width increases, and most softwoods compress more than dense hardwoods. Also, warped laminations result in more compression as the laminations are straightened during the stressing operation.

For the bridges evaluated in the monitoring program, the most frequent construction problems, insufficient prestress, deck distortion, and deck attachment damage, were encountered during the stressing procedure.

Insufficient Prestress

For acceptable bridge performance, all bars must be uniformly stressed to the full design level during each of the three required stressings. Field observations indicate that, when a single jack is used, stressing one bar compresses the deck at that location and reduces the force in adjacent bars. In bridges where each bar was stressed only one time, substantial variations in bar force were noted. To prevent these variations, each bar must be stressed several times as the deck compresses until the prestress level is uniform for all bars. The most successful construction method for accomplishing this uniformity is to begin stressing at one bridge end and sequentially stress each bar along the bridge length. This process is repeated until three to six passes have been made along the bridge length and the force reaches the designated level in all bars.

Deck Distortion

Compression of the laminations during stressing led to deck distortion in numerous bridges. To keep the bridge edges parallel and straight, the initial bridge stressing must be gradual, starting at a low prestress that is gradually increased. If the full prestress level is placed initially in one bar, the deck will compress significantly at that location and deck distortion can result. Field observations have shown that this can lead to an hourglass shape (the bridge is narrower at centerspan than at the ends) if the midspan bar is fully stressed first or a reverse hourglass shape (bridge ends are narrower than centerspan) if the end bars are fully stressed first. Other patterns of distortion have also been observed. To minimize deck distortion during the initial stressing, field observations indicate that a low initial prestress of 10 to 25 percent of the design level should be placed in the bars and the deck shape observed. If distortion of the deck is evident, the prestress is adjusted accordingly until the distortion is removed. The prestress is then increased to 25 to 50 percent of the design level, and the bridge is restressed again. This process is repeated until the deck is completely stressed to the full design level.

Deck Attachment Damage

When attachments to the bridge deck are made before deck narrowing, damage to the deck and attachments may occur. This damage is most evident when curbs are bolted in-place or the bridge is attached to the substructure before stressing is complete. As a result, fasteners and other metal components may bend, and wood may be damaged. To prevent such damage, field observations indicate that deck attachments should be made only after the second bridge stressing is complete.

Moisture Content

The moisture content of wood at installation and in-service is a primary consideration for the design of all exposed wood structures. Changes in moisture content can affect strength, stiffness, and dimensional stability. Changes in strength and stiffness are recognized in design by applying wet-use reductions to design values, when applicable. Of primary concern in stress-laminated bridges is the dimensional stability of the wood as changes in moisture content occur. Below the fiber saturation point (approximately 30 percent), wood will expand as moisture is gained and contract when moisture is lost. In stress-laminated bridges, these dimensional changes can affect bridge performance. Moisture content changes in stress-laminated decks generally can be considered as global changes and localized changes. Global changes affect the entire structure and occur slowly as the moisture content of the bridge laminations at the time of construction moves toward an equilibrium moisture content with the environment (*4*). Localized changes affect the exposed portions of the bridge and occur more rapidly in response to surface wetting or seasonal fluctuations in equilibrium moisture content.

Global Moisture Content Effects

The effect of global moisture content changes in stress-laminated timber bridges depends on the moisture content of the wood laminations at the time of construction and the average equilibrium moisture content for the bridge site. With few exceptions, bridges included in the monitoring program were made of sawn lumber with a relatively high moisture content. Typical moisture content values at the time of construction ranged from 25 to 29 percent; however, moisture contents in excess of 30 percent have been measured on numerous bridges. At such high levels, the wood moisture content substantially exceeds the expected equilibrium moisture content, which typically averages 16 to 20 percent depending on the bridge location (*12*). Conversely, several stress-laminated timber bridges constructed with glulam laminations have been installed with average moisture contents as low as 12 percent. Field measurements have shown that global moisture content changes toward an equilibrium level are relatively slow. As a result, the observed effects of global moisture content changes are minimal during the first several months after bridge construction. However, the effects become much more pronounced as the decks eventually lose or gain moisture. Such global moisture content changes directly affect stressing bar force levels, which decrease when moisture is lost and increase when moisture is gained. Based on field evaluations, the best bridge performance has been observed when the moisture content of the wood laminations at the time of construction averages 10 to 16 percent. Acceptable performance has been observed when the moisture content is 16 to 20 percent. As the moisture content increases above 20 percent at the time of construction, adverse performance becomes more pronounced as the moisture content is increased.

Localized Moisture Content Effects

Field data indicate that localized moisture content changes caused by surface wetting and seasonal moisture content changes can also affect the performance of stress-laminated timber bridges. The most pronounced affect appears to occur in relatively deep decks, 300 to 400 mm (12 to 16 in.) thick, when the top deck surface is exposed or covered with a lumber plank wearing surface. In such cases the deck surface absorbs moisture more rapidly than the inner and lower portions. As a result, repeated wetting or standing water may cause the upper portion of the deck volume to swell in relation to the lower portion. Although no adverse structural effects have resulted from this response, evidence of differential moisture content is observed as a slight transverse crown in the deck, wood crushing in the outside edge laminations along the top of the bar anchorages, and/or an increase in stressing-bar force. The comparative performance of several bridges indicates that the potential for these conditions can be greatly reduced or eliminated if the deck surface is paved with asphalt, preferably in combination with a waterproof geotextile membrane.

Stressing-Bar Force

The structural integrity and serviceability of stress-laminated decks depends on the compressive stress maintained among the lumber laminations. For acceptable performance, this compression must be sufficient to prevent vertical slip because of shear and opening between the laminations because of transverse bending. Current design procedures recommend a minimum interlaminar compression of 690 kPa (100 lb/in.^{2}) at the time of bridge construction. This initial compressive stress is based on the assumption that 50 to 60 percent of the stress will be lost over the life of the structure because of wood stress relaxation and minor changes in wood moisture content (*4*). Research has shown that slip between the laminations does not begin until the interlaminar compression has been reduced to 140 to 165 kPa (20 to 24 lb/in.^{2}).

As previously discussed, construction procedures for stress-laminated timber bridges recommend that bridges be stressed three separate times over a period of six to eight weeks. Based on monitoring program results, it appears that this stressing sequence is not adequate in many cases. Many of the bridges in the monitoring program required restressing within the first two years after construction. For bridges constructed of sawn lumber, field observations indicate that the bar force should be checked at annual intervals for the first 2 years after construction and every 2 years thereafter. After bar force stabilizes, this period may be extended to 2- to 5-year intervals. For bridges constructed of glued laminated timber, field observations indicate that bar force should be checked every 2 years for the first 4 years after construction and every 5 years thereafter. These observations are based on the behavior of numerous bridges and should be adjusted for site specific conditions.

Of the bridges included in the monitoring program, bar force loss resulted in structural problems on one bridge. In this case it was known that the bar force was rapidly dropping, yet no corrective action was taken to restress the bridge. Vertical slip of the laminations resulted from heavy truck traffic and was evident in one lane at centerspan as a depression where truck wheel lines were tracking. After the slip occurred, the bridge continued to carry traffic at a reduced load level until it was restressed and subsequently repaired. When slip of this type occurs, the stressing bars act as dowels among the laminations, and the initial failure primarily affects serviceability and is very evident. Thus, ample warning is given so that appropriate repair can be made before further problems develop.

For monitoring purposes, compressive stress among the laminations is determined by measuring the stressing-bar force. Field performance of stress-laminated bridges has shown that bar force, and thus interlaminar compression, is a complex interaction of many effects including wood stress relaxation, moisture content changes, bar anchorage performance, and temperature fluctuations. When evaluating the causes of loss of stressing-bar force for field bridges, it is impossible to determine the individual effect of the numerous contributing factors accurately. However, we can make the following observations relative to the general performance of bridges included in the monitoring program.

Stress Relaxation

When laminations are subjected to the long-term loads applied by stressing bars, the wood slowly deforms over the entire bridge width, and the bar force is reduced. This phenomenon, known as stress relaxation, is similar to creep. The rate of stress relaxation is greatest when the bridge is initially stressed and normally decreases with each subsequent stressing. Field observations indicate that bar force loss because of stress relaxation continues at a slow rate that gradually decreases after construction, depending on several factors. Stress-relaxation losses increase as the moisture content of the wood increases and are greater for softwoods such as Douglas fir and pine than for dense hardwoods such as oak or maple. In addition, bar force loss because of stress relaxation increases as the bridge width increases (the volume of wood between the bar anchorages increases).

Moisture Content

The moisture content at the time of construction is one of the most influential factors on maintaining bar force. The best performance occurs when the wood laminations are installed at an average moisture content less than 16 percent. At this low level, global increases in lamination moisture content toward an equilibrium level result in swelling, which increases bar force, offsets loss because of stress-relaxation, and is beneficial. When installed at moisture contents above 20 percent but less than 30 percent, moisture content decreases are gradual but result in a loss in bar force of as much as 80 percent over an 18 month period. When lumber laminations are above the fiber saturation point at the time of construction, drying is slow. At such high moisture contents, no loss in bar force because of wood shrinkage is evident until the wood dries below the fiber saturation point; however, as the moisture content decreases below fiber saturation, bar force losses are substantial.

Anchorage System Performance

The purpose of the anchorage system for stressing bars is to distribute the bar force into the deck without causing wood crushing along the outside laminations. When crushing does occur, force reduction in the stressing bars can be substantial. Historically, anchorage systems used for stress-laminated decks have used a steel channel or discrete plate configuration (Figure 3). The channel configuration was developed in Ontario and is currently a design requirement of the OHBDC. The discrete plate was developed in the United States and uses a single rectangular bearing plate at each bar. With few exceptions, the bridges included in the monitoring program use the discrete plate anchorage because it is less expensive than the steel channel and provides acceptable performance when properly designed. When properly sized plates are used on softwood lumber species (Douglas fir and pine), crushing into the bridge outside laminations has typically averaged 3 to 6 mm (1/8 to 1/4 in.). On dense hardwood laminations, such as oak and maple, properly designed plates show virtually no crushing. Field observations indicate that anchorage performance on softwood bridges is improved when two more dense hardwood laminations are used along the bridge edges.

Thermal Response

Bridges included in the FPL monitoring program are located throughout the United States. Numerous bridges are subjected to annual temperature variations of 38°C (100°F) or more. Two bridges have been instrumented to measure the effect of large temperature decreases on the stressing bar force. These bridges are located in regions where ambient temperatures have reached 38°C (100°F) in the summer and −40°C (−40°F) in the winter. Data collected for both bridges indicate that bar force decreases when temperature drops. The magnitude of this decrease depends on a number of factors including the magnitude of the temperature change, the duration of cold temperature, the wood species, and the moisture content. The temperature effect is most pronounced when the wood moisture content is at or above fiber saturation. Short-term temperature declines over periods of 24 hours or less have little effect on bar force because thermal conductivity of wood is very low. The cold temperature effect appears to be fully recoverable, and the bar force returns to the original level when the temperature is increased. At this time, no evidence indicates that temperature effects alone result in any structural or serviceability problems in stress-laminated timber bridges; however,

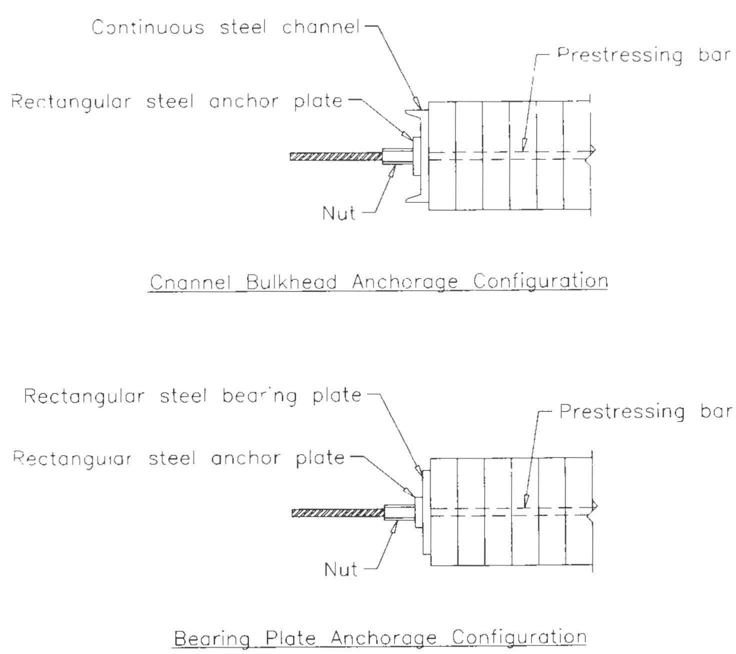


FIGURE 3 Stressing bar anchorage configurations commonly used on stress-laminated bridge decks.

extensive laboratory and field work in this area is continuing, and more definitive conclusions will be forthcoming.

Vertical Creep

As a structural material, wood is subject to permanent deformation because of long-term sustained loads. This deformation, known as creep, depends on a number of factors and is more pronounced when the magnitude of applied loading and the moisture content of the wood increase. For timber bridges, creep results in vertical deformation of the bridge span and, in extreme cases, a noticeable sag. Although this is not a significant structural problem, a sagging bridge is alarming to the public. In addition, creep can disrupt bridge drainage and facilitate water ponding, which may be a hazard to bridge users. To offset the effects of creep, stress-laminated timber bridges made with glulam timber or lumber with butt joints can be cambered; however, bridges made with continuous lumber cannot be cambered.

In general, creep has not been a problem in stress-laminated timber bridges. Three bridges, which were among the first built in the United States, are exceptions in that creep caused a sag at centerspan of 50 to 75 mm (2 to 3 in.). In each case, the bridges have a high span/depth ratio, were installed with a high lamination moisture content greater than 28 percent, and had butt joints.

In comparing the behavior of these bridges to that of other stress-laminated timber bridges, the most important factor affecting creep appears to be the magnitude of the permanent load bending stress. Although the moisture content of the laminations and the spacing and frequency of butt joints also affect creep, the relative contribution of these factors compared to permanent load bending stress appears to be small. Currently, there are no design provisions for limiting creep; however, span limitations resulting from live load deflection requirements appear to be sufficient to limit permanent load bending stress adequately and control creep to an acceptable level. Creep has not been a detectable problem in stress-laminated bridges when the live load deflection for standard highway loading is limited to 1/360 to 1/400 of the bridge span, regardless of the presence of butt joints or the moisture content of the laminations.

Load Test Behavior

Load tests were conducted on all stress-laminated timber bridges in the monitoring program to assess structural behavior under a static loading condition. Each bridge is load tested twice: shortly after construction and at the end of the monitoring period, 2 to 3 years later. Additional load tests may be conducted if the further evaluation of unique features is considered beneficial. In addition to static tests, dynamic load tests were also conducted on nine bridges. In all cases, load tests showed that stress-laminated timber decks act as large orthotropic plates. The magnitude of the deck displacements and the deformed shape of the loaded bridge depend on a number of factors including the bridge span and width, vehicle weight and configuration, deck material properties, the location and frequency of butt joints, the prestress level, and the edge stiffening effects of curb and rail systems. Based on results from the static and dynamic tests and analytical modeling, revised methods for predicting the behavior of stress-laminated timber bridges are currently being developed at FPL.

Condition Evaluation

The condition of each bridge in the monitoring program is evaluated several times during the monitoring period. In the course of these evaluations, information is collected relative to the general bridge condition, stressing system corrosion, and asphalt wearing surface performance.

General Condition

General condition assessments are performed on stress-laminated timber decks to evaluate the performance of various components and design features unrelated to the features associated with stress-laminating. The majority of the noted deficiencies have been relatively minor but typically could develop into more serious problems as time passes without corrective action. In general, most deficiencies are directly attributable to poor design detailing and/or construction methods. Unfortunately, many of the same deficiencies are also common on other types of timber bridges primarily because of the inexperience of most engineers and contractors in wood design and construction methodology. As time progresses and more information on timber bridges becomes available, engineers and contractors will become more experienced, and these problems should be minimized.

Several of the common observations related to the general condition of stress-laminated timber decks are summarized below.

1. *Increased emphasis is needed in the area of field treating.* Ideally, all wood used in bridge construction should be completely fabricated before it is treated with wood preservatives. This method is difficult for stress-laminated decks because the location of bolt holes for

deck attachments cannot be confirmed until the bridge is stressed. Consequently, deck holes for curbs, railing, and substructure attachment must often be field drilled. When this is done, the exposed untreated wood must be field treated with wood preservative. On numerous bridges, evidence of field drilling and cutting without field treatment has been evident. This method exposes untreated wood and can lead to premature deterioration. Proper field treating in accordance with American Wood Preservers' Association (AWPA) Standard C14 (*13*) will significantly reduce deterioration potential.

2. *Improved design detailing and maintenance are needed for debris control.* Accumulations of dirt and debris on wood bridges can trap moisture and create an environment suitable for deterioration. Although wood preservatives effectively protect the wood, deterioration is possible when field fabrication has been used or the preservative treatment is incomplete. On many of the stress-laminated bridges in the monitoring program, significant debris accumulations were observed on the bridge deck, under curb openings, and at the bearings. Although no adverse effects were noted, the potential for future deterioration was evident. To some degree, debris accumulation can be reduced by proper design detailing. However, periodic maintenance to remove debris is essential for maximizing bridge performance and longevity.

3. *Special attention is needed to ensure proper wood treatment.* Wood used in stress-laminated decks is typically treated with oil-type preservatives in accordance with AWPA Standard C14 (*13*). Dripping of the preservative from the bridge has not been a widespread problem; however, minor preservative dripping has been observed on several bridges. In such cases, the bridges were treated to preservative retentions substantially above those required by AWPA standards. The subsequent compression among the laminations because of stressing forced minor amounts of preservative from the laminations. Preservative dripping does not appear to be a problem when the laminations are properly treated in accordance with AWPA stadards to the required preservative retention.

Stressing System Corrosion

Adequate corrosion protection of the steel stressing system has been a primary consideration since the development of stress-laminated timber bridges. The original bridges constructed in Ontario used a plastic tube filled with grease to protect the stressing bars. Bridges built in the United States have typically used galvanizing as a means of corrosion protection, although several bridges have been built with galvanized bars placed in grease-filled tubes. Over the past 6 years of the bridge monitoring program, corrosion has occasionally been observed in exposed bar locations where anchorage nuts were not oversized to compensate for bar galvanizing. This corrosion resulted when the nuts were forced onto the bars during construction and the galvanizing was damaged. Properly sizing nuts or applying a cold galvanizing compound to the damaged areas will eliminate this problem. At interior bar locations inside the deck, bar corrosion has not been a problem; however, the monitoring period has been relatively short and definitive conclusions on long-term corrosion potential cannot be made. Based on preliminary observations, enclosing the bars in grease-filled plastic tubes may be warranted if the bridge is subjected to corrosive deicing chemicals in winter months. In addition, protective tubes may be warranted when the lumber laminations are treated with waterborne preservatives containing copper and it is anticipated that the lamination moisture content will exceed 20 percent. Under these conditions, depletion of zinc in the galvanizing is possible because of an electrochemical reaction with copper in the wood preservative.

Asphalt Wearing Surface Performance

The performance of asphalt wearing surfaces on wood bridge decks has long been a concern of bridge engineers. In the past, several wood deck systems employing nail-laminated lumber or non-interconnected deck panels have been associated with cracking or disintegration of asphalt wearing surfaces. This deterioration is caused by differential movement among individual laminations or vertical movement at joints. Many of the stress-laminated timber bridges in the monitoring program were paved with an asphalt wearing surface. In most cases, the asphalt was placed to a compacted thickness of 50 to 75 mm (2 to 3 in.) at centerline and tapered to a compacted thickness of approximately 40 mm (1.5 in.) along the deck edges. Because stress-laminated decks act as large wood plates and the applied prestress sufficiently prevents vertical movement of the individual laminations, asphalt cracking and deterioration related to bridge performance were not observed on any of the stress-laminated decks. Even on decks designed for full highway loads with a design live-load deflection as high as 1/250 of the bridge span, no asphalt cracking or deterioration has been apparent during the monitoring period.

Summary

Several hundred stress-laminated timber bridges have been built in the United States since 1988. Based on observations of 24 bridges that were monitored over a period of 2 years or more, bridge performance has generally been satisfactory although performance can be improved in several areas. Key recommendations based on monitoring program observations follow.

1. When bridges are stressed with a single jack, three to six stressing passes should be made along the bridge length to ensure uniform prestress at the required level. In addition, the stress level should be gradually increased over the first several passes to minimize deck distortion.

2. Attachments to the bridge superstructure including curbs, railings, and substructure attachments should not be made until after the bridge has been fully stressed two times.

3. The average moisture content of the wood laminations at the time of bridge construction should preferably be 10 to 16 percent but should not exceed 20 percent.

4. For bridges constructed of sawn lumber, bar force should be checked at annual intervals for the first two years after construction and every two years thereafter. This period may be extended after bar force stabilizes to 2- to 5-year intervals. For bridges constructed of glued laminated timber, bar force should be checked every 2 years for the first 4 years after construction and every 5 years thereafter.

5. Bridge live-load deflection should be limited to a maximum of 1/360 to 1/400 of the bridge span.

6. When oil-type wood preservatives are used, the preservative retention should not exceed that recommended in AWPA Standard C14 (*13*).

7. Consideration should be given to enclosing stressing bars in grease-filled plastic tubes if the bridge is subjected to corrosive deicing chemicals or if the lumber laminations are treated with waterborne preservatives containing copper and it is anticipated that the lamination moisture content will exceed 20 percent.

REFERENCES

1. Taylor, R. J., and P. F. Csagoly. *Transverse Post-Tensioning of Longitudinally Laminated Timber Bridge Decks*. Ministry of Transportation and Communications, Downsview, Ontario, Canada, 1979.
2. Taylor, R. J., B. Batchelor, and K. Van Dalen. *Prestressed Wood Bridges*. SRR-83-01. Ministry of Transportation and Communications, Downsview, Ontario, Canada, 1983.
3. Ministry of Transportation and Communications. *Ontario Highway Bridge Design Code*. Downsview, Ontario, Canada, 1983.
4. Ritter, M. A. *Timber Bridges: Design, Construction, Inspection, and Maintenance*. EM 7700-8. USDA Forest Service, Washington, D.C., 1990.
5. Davalos, J. F., and S. H. Petro. *Design, Construction and Quality Control Guidelines for Stress-Laminated Timber Bridge Decks*. Report FHWA-RD-91-120. FHWA, U.S. Department of Transportation, 1993.
6. *Guide Specifications for the Design of Stress-Laminated Wood Decks*. AASHTO, Washington, D.C., 1991.
7. Ritter, M. A., E. A. Geske, W. J. McCutcheon, R. C. Moody, J. P. Wacker, and L. E. Mason. Methods for Assessing the Field Performance of Stress-Laminated Timber Bridges. In *Proceedings of the 1991 International Timber Engineering Conference*, September 2–5, 1991, TRADA, London, 1991, Vol. 3, pp. 3.319–3.326.
8. Wacker, J. P., and M. A. Ritter. *Field Performance of Timber Bridges: 1. Teal River Stress-Laminated Deck Bridge*. Res. Pap. FPL-RP-515. Forest Products Laboratory, USDA Forest Service, Madison, Wisc., 1992.
9. Dickson, B. D., and H. V. S. GangaRao. *Field Monitoring of the Stressed Timber Bridge Over Elk Two Mile Creek, WV*. West Virginia University, Morgantown, 1989.
10. Gutkowski, R. M., and S. L. Lewis. *Monitoring and Testing of a Prestressed Longitudinal Deck Timber Bridge*. Colorado State University, Ft. Collins, 1989.
11. Mozingo, R. R., and G. DiCarlantino. *Trout Road Bridge Project*. Pennsylvania State University, State College, 1988.
12. McCutcheon, W. J., R. M. Gutkowski, and R. C. Moody. Performance and Rehabilitation of Timber Bridges. In *Transportation Research Record 1053*, TRB, National Research Council, Washington, D.C., 1986, pp. 65–69.
13. AWPA. *Book of Standards*. American Wood Preservers' Association, Woodstock, Md., 1993.

Design and Evaluation of Two Bridge Railings for Low-Volume Roads

Ronald K. Faller, Barry T. Rosson, and Dean L. Sicking, *University of Nebraska-Lincoln*
Michael A. Ritter, *Forest Products Laboratory, USDA Forest Service*
Steve Bunnell, *USDA Forest Service, Washington, D.C.*

The U.S. Department of Agriculture (USDA) Forest Service, Forest Products Laboratory (FPL) and Headquarters Engineering Staff, in cooperation with the Midwest Roadside Safety Facility undertook the task of developing bridge railing systems for roads with low traffic volumes and low speeds. Two low-cost bridge railing systems were developed and successful full-scale crash tests were conducted for their use on timber bridge decks using longitudinal lumber laminations. A curb-type timber railing system was designed to redirect a 3/4-ton pickup truck hitting at a speed of 24 km/hr (15 mph) and an angle of 15 degrees. The curb-type rail system used square, trapezoidal, or rectangular rail shapes. A flexible railing system consisting of steel W-beam supported by breakaway timber posts was designed to redirect a 3/4-ton pickup truck hitting at a speed of 50 km/hr (31 mph) and an angle of 25 degrees. The flexible railing system was developed according to Test Level 1 of NCHRP Report 350, *Recommended Procedure for the Safety Performance Evaluation of Highway Features.*

Historically, bridge railing systems have not been developed for use on low-speed, low-volume roads; however, many U.S. Forest Service and National Forest utility and service roads often carry very low traffic volumes at operating speeds of 24 to 32 km/hr (15 to 20 mph) or less. These roads are often narrow, generally incorporating one- or two-lane timber bridges with span lengths between 4.6 and 10.7 m (15 and 35 ft). The bridge rails that have been designed for high-speed facilities may be too expensive for low-volume roads. In recognition of the need to develop bridge railings for this very low service level, the U.S. Department of Agriculture (USDA) Forest Service, Forest Products Laboratory (FPL) and Headquarters Engineering Staff, in cooperation with the Midwest Roadside Safety Facility (MwRSF), undertook the task of developing two bridge railing systems.

Objective

The objective of this research project was to develop two low-cost bridge railing systems for use on longitudinal timber bridge decks with low traffic volumes and speeds. A longitudinal glulam timber deck was selected for use in the development of the bridge railings because it is the weakest type of longitudinal timber deck for resisting transverse railing loads currently in use. Thus, any bridge railing not damaging the longitudinal glulam deck could be easily adapted to other, stronger, timber deck systems.

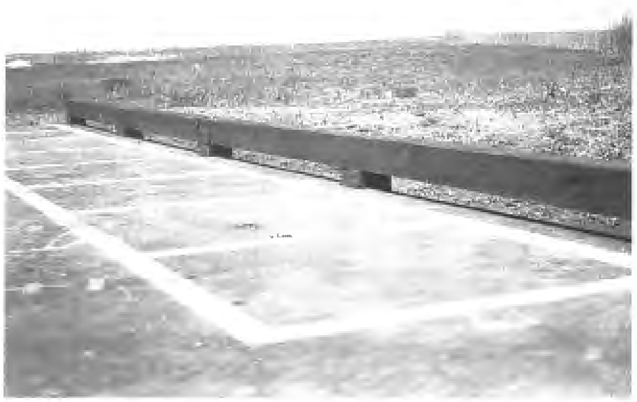
Curb-type railing systems were chosen as the basic design for the first bridge railing. A top-mounted curb-type railing is shown in Figure 1(*a*). Although curb barriers generally offer limited redirective capability at higher impact speeds, curb barriers can be very effective during low-speed impacts. A flexible railing with a breakaway post system was selected as the basic design for the second bridge railing. A side-mounted flexible railing is shown in Figure 1(*b*).

Evaluation Criteria

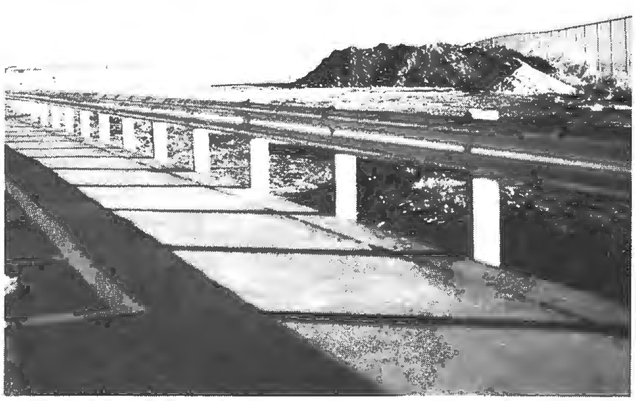
Background

Currently, bridge railings are usually designed to satisfy the requirements provided in AASHTO's *Guide Specifications for Bridge Railings* (*1*). More specifically, bridge railings should be designed according to the appropriate performance level of the roadway, based upon a number of factors such as design speed, average daily traffic (ADT), percentage of trucks, bridge rail offset, and number of lanes. These guide specifications include three performance levels, shown in Table 1, which provide criteria for evaluating the safety performance of bridge railings.

The recently published NCHRP Report 350, *Recommended Procedure for the Safety Performance Evaluation of Highway Features* (*2*), provides for six test levels, shown in Table 1, for evaluating longitudinal barriers. Although this document does not contain objective criteria for selecting test level, safety hardware developed to meet the lower test levels is generally intended for use on lower-service-level roadways, and higher-test-level hardware is intended for use on higher-service-level roadways. The lowest performance level, Test Level 1, is suitable for applications on low-volume, low-speed facilities such as residential streets. However, operating speeds on these facilities are typically in the range of 48 km/hr (30 mph) or approximately twice as high as operating speeds on Forest Service utility roads. Thus, test impact conditions from Test Level 1 were deemed too severe for the low-cost curb-type bridge railing system envisioned. The second bridge railing, or flexible railing, was designed to meet Test Level 1 impact conditions because the increase in performance level could be achieved with little increase in cost.



(a)



(b)

FIGURE 1 (*a*) Curb-type bridge railing and (*b*) flexible bridge railing.

Crash Test Conditions

Design impact conditions for narrow, low-volume utility roads were selected by the Forest Products Laboratory (FPL) of the USDA Forest Service in consultation with engineers of the Headquarters Engineering Staff. Reasonable design impact conditions for the curb-type bridge railings were estimated to involve a 3/4-ton pickup truck hitting at a speed of 24 km/hr (15 mph) and an angle of 15 degrees. The design impact conditions for the flexible bridge railing involved a 3/4-ton pickup truck hitting at a speed of 50 km/hr (31 mph) and an angle of 25 degrees according to Test Level 1 in NCHRP Report 350 (*2*). It is noted that a research study is in progress to develop a curb-type bridge railing to meet Test Level 1 of NCHRP Report 350 (*2*).

Curb-Type Bridge Railings

Design Considerations

Timber was selected for use in the curb-type bridge railing designs on the basis of aesthetics and material availability. Further, curb railings were identified as a low-

TABLE 1 AASHTO Crash Test Conditions for Bridge Railings and NCHRP Report 350 Crash Test Conditions for Longitudinal Barriers

AASHTO Performance Level (1)	Impact Conditions				
	Small Car (816 kg)	Pickup Truck (2,449 kg)	Medium Single-Unit Truck (8,165 kg)	Van-Type Tractor-Trailer (22,680 kg)	
1	80.5 km/h and 20 deg	72.4 km/h and 20 deg			
2	96.6 km/h and 20 deg	96.6 km/h and 20 deg	80.5 km/h and 15 deg		
3	96.6 km/h and 20 deg	96.6 km/h and 20 deg		80.5 km/h and 15 deg	
NCHRP 350 Test Level (2)	Impact Conditions				
	Small Car (820 kg)	Pickup Truck (2,000 kg)	Single-Unit Van Truck (8,000 kg)	Tractor/Van Trailer (36,000 kg)	Tractor/Tank Trailer (36,000 kg)
1	50 km/h & 20 deg	50 km/h & 25 deg			
2	70 km/h & 20 deg	70 km/h & 25 deg			
3 (Basic Level)	100 km/h & 20 deg	100 km/h & 25 deg			
4	100 km/h & 20 deg	100 km/h & 25 deg	80 km/h & 15 deg		
5	100 km/h & 20 deg	100 km/h & 25 deg		80 km/h & 15 deg	
6	100 km/h & 20 deg	100 km/h & 25 deg			80 km/h & 15 deg

cost railing system and the most easily constructed design alternative for these low-service-level applications. Since most economical timber curb systems incorporate top-mounted single-railing designs, this type of structure was used for the new bridge rails.

Analysis of vehicular impacts with concrete and timber curbs revealed that the shape of the curb face could affect the redirective capacity of curb systems. A number of curb shape configurations were included in the design process. Each curb configuration was evaluated at different heights in order to determine the minimum height required to meet the selected performance criteria. Based on full-scale vehicle crash tests of curb systems 50.8 cm (20 in.) high (6) and a limited study of impacts with shorter curbs (unpublished research) using HVOSM computer simulation modeling (7), the researchers estimated that curbs 20.3 to 35.6 cm (8 to 14 in.) high should be able to meet the desired performance standard.

Peak lateral forces imparted to the curb railing were estimated to be approximately 9.5 kN (2.1 kips) using the procedures described by the NCHRP report, the AASHTO Guide, and Ritter et al. (3–5). Based on these findings, it was concluded that timber curb railings may be capable of withstanding design impact conditions without significant damage to the barrier or the timber deck. Each railing was analyzed as a simply supported beam with pin connections at each end. Three rail shapes and sizes—a 20.3-cm (8-in.) by 20.3-cm (8-in.) square, a 20.3-cm (8-in.) by 22.9-cm (9-in.) trapezoid with a negative slope on the traffic-side face, and a 10.2-cm (4-in.) by 30.5-cm (12-in.) rectangle—were selected for a preliminary evaluation. A developmental testing program was then undertaken to evaluate the safety performance and height requirements for each of these curb rails.

Design Details

The basic curb design incorporated 6.10-m (20-ft) long rail sections mounted on scupper blocks. The rail elements, scupper blocks, and bridge deck were attached to each other with two 1.6-cm (5/8-in.) diameter ASTM A307 galvanized bolts placed 15.2 cm (6 in.) apart at each end and in the middle of each rail element. A bolted lap splice was also incorporated to attach the ends of adjacent rail elements. The 11.9-m (39-ft) long curb rails were constructed from two 6.10-m (20-ft) long rail sections and a 0.30-m (1-ft) long lap splice. Two sizes of timber scupper blocks were used to mount the curb rail elements on the timber deck. The curb rail sections and scupper blocks were constructed from No. 1 Grade Douglas fir using rough-sawn and SIS specifications, respectively. Timber curb rail and scupper materials were treated to meet AWPA Standard C14 with 192.22 kg/m^3 (12 pcf) creosote (*8*). Schematics of both a typical curb rail section mounted on the deck surface and a curb railing splice are shown in Figure 2.

The curb railings were attached to a longitudinal glulam timber deck supported by concrete abutments. The concrete abutments and the longitudinal glulam timber deck were the same as those used in the development of previously tested AASHTO PL-1 and PL-2 railing systems (*9–11*). In addition, a 5.1-cm (2-in.) asphalt surface was placed on the top of the timber deck in order to represent actual field conditions.

Developmental Testing, Phase I

Developmental testing was used to determine critical heights for the three different curb shapes. The developmental testing used a 1985 Ford F-250 3/4-ton pickup truck with test inertial and gross static weights of 1999 kg (4,406 lb) and 2078 kg (4,581 lb), respec-

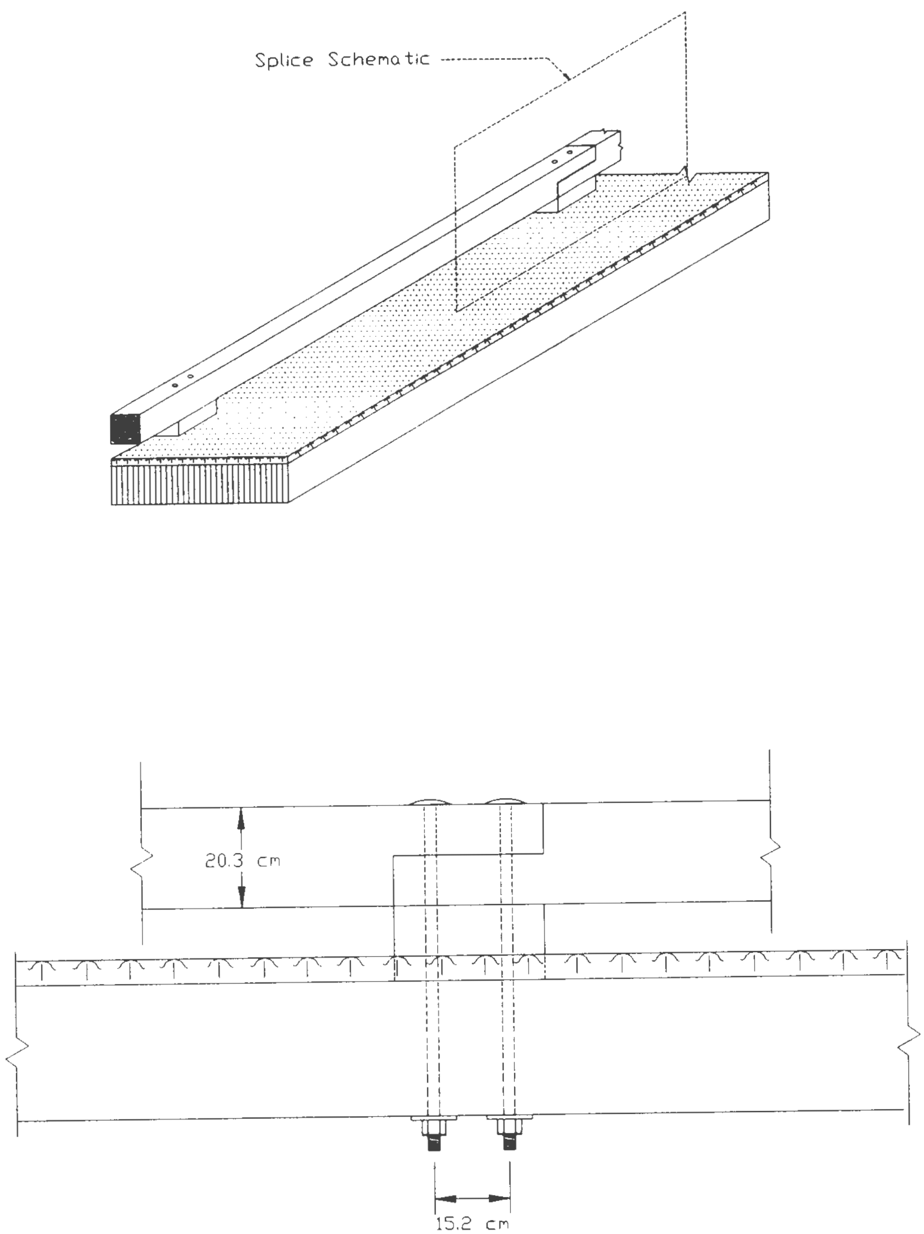


FIGURE 2 **Typical curb section mounted to deck surface and curb railing splice.**

tively. A pickup truck was driven into the rails at speeds of 24 and 32 km/hr (15 and 20 mph) and an angle of 15 degrees. No steering or braking inputs were applied to the vehicle during impact or until the vehicle had traveled an adequate distance downstream from the end of the rails.

The curb shapes were attached to a continuous concrete slab, as shown in Figure 3, with two 1.6-cm (5/8 in.) diameter ASTM A307 bolts spaced on 2.90-m (9-ft 6-in.) centers. If necessary, timber scupper blocks were placed below the rail shapes in order to mount the curb rails 20.3, 25.4, and 30.5 cm (8, 10, and 12 in.) above the surface.

Impact tests were performed on the three curb shapes mounted at three different heights for a total of nine curb configurations. The developmental testing phase consisted of 19 impact tests on the rail attached to the concrete slab, as shown in Table 2. For impacts at 24 km/hr (15 mph) and 15 degrees, the trapezoidal and rectangular shapes with a 20.3-cm (8-in.) mounting height successfully redirected the test vehicle with no tendency for the vehicle to climb. However, for the same impact conditions, the square shape with a 20.3-cm (8-in.) mounting height allowed the vehicle to climb over the top of the rail. Following these tests, it was determined that one full-scale vehicle crash test would be performed on one of the two successful curb shapes attached to the longitudinal timber deck. The trapezoidal shape with a 20.3-cm (8-in.) mounting height was selected for this crash testing because it appeared to provide a higher redirective capacity than the rectangular shape.

Full-Scale Crash Testing, Phase I

Full-scale crash testing used the same 3/4-ton pickup truck but with a test inertial and gross static weight of 1999 kg (4,406 lb), an impact speed of 24 km/hr (15 mph), and an angle of 15 degrees. The test vehicle was towed using a cable tow and guidance system and struck the rails attached to the longitudinal timber deck.

Originally, only one full-scale crash test was to be conducted on a 20.3-cm (8-in.) by 22.9-cm (9-in.) trapezoidal shape with 20.3-cm (8-in.) mounting height. However, because this test failed, two additional tests were conducted on the trapezoidal shape, one at the 20.3-cm (8-in.) mounting height and one at the 25.4-cm (10-in.) mounting height.

In Test LVCT-1a the vehicle struck the curb rail approximately 3.35 m (11 ft) from the upstream end of the 11.9-m (39-ft) long installation. During impact, the vehicle's right front tire climbed over the top of the curb. The vehicle came to rest on top of the curb at the end of the installation. In Test LVCT-lb the vehicle struck the curb rail at the same location as in Test LVCT-1a. The vehicle's right front tire again climbed over the curb with little or no vehicle redirection. Following the two unsuccessful tests on the trapezoidal shape with a 20.3-cm (8-in.) mounting height, a third test was conducted on the trapezoidal shape with a 25.4-cm (10-in.) mounting height. The impact point for Test LVCT-1c was the same as that for the previous two tests. The vehicle's right front tire again climbed over

FIGURE 3 Square curb rail attached to concrete apron, three views.

TABLE 2 Summary of Curb-Type Bridge Railing Development, Phase I

Rail Type (cm x cm)	Rail Height (cm)	Test No.	Speed (km/h)	Results
Square - 20.3 x 20.3	20.3	1	24	Failed - vehicle over top of curb
Square - 20.3 x 20.3	25.4	2a 2b	24 24	Passed - right front tire briefly hopped onto curb Passed - right front tire popped into air
Trapezoid - 20.3 x 22.9	25.4	3a 3b 3c 3d	24 24 32 32	Passed - no climbing tendency Passed - no climbing tendency Passed - right-front tire popped into air Passed - right-front tire briefly hopped onto curb
Trapezoid - 20.3 x 22.9	20.3	4a 4b 4c	24 24 32	Passed - no climbing tendency Passed - no climbing tendency Failed - vehicle over top of curb
Trapezoid - 20.3 x 22.9	30.5	5	24	Passed - no climbing tendency
Square - 20.3 x 20.3	30.5	6a 6b	24 24	Passed - no climbing tendency Passed - no climbing tendency
Rectangle - 10.2 x 30.5	30.5	7a 7b	24 24	Passed - no climbing tendency Passed - no climbing tendency
Rectangle - 10.2 x 30.5	25.4	8a 8b	24 24	Passed - no climbing tendency Passed - no climbing tendency
Rectangle - 10.2 x 30.5	20.3	9a 9b	24 24	Passed - no climbing tendency Passed - no climbing tendency

the top of the curb, which allowed the tire to go over the side of the bridge rail. The vehicle came to rest on top of the curb at the end of the installation.

Results of these tests were inconsistent with the previous findings from the developmental testing program. Factors that may have affected the results include the following: (*a*) air temperatures were much warmer when testing on the timber deck than during developmental testing on the concrete slab (average daily temperatures during developmental testing, Phase I, and full-scale crash testing, Phase I, were −2.2°C (28°F) and 17°C (63°F), respectively); (*b*) the trapezoidal curb rail was coated with a latex water-based paint to aid in photography and documentation of tests; and (*c*) creosote on the surface of the treated timber may have dried and increased friction levels between the tires and timber rail.

Developmental Testing, Phase II

Following three unsuccessful full-scale vehicle crash tests on the longitudinal deck with the trapezoidal curb rail, developmental testing was once again conducted on the concrete slab to determine the critical mounting heights for the three different curb shapes. The curb shapes were attached to the concrete in the same manner as during the first phase of the developmental testing program. The 3/4-ton pickup truck was again driven into the curb railings at a speed of 24 km/hr (15 mph) and an angle of 15 degrees. The trapezoidal shape rail was tested with the same coating of paint used during the full-scale crash tests and creosote that may have dried on the timber rail surface.

Impact tests were performed on the three curb shapes mounted at heights ranging from 20.3 to 35.6 cm (8 to 14 in.) A total of eight curb configurations were evaluated with 15 crash tests, as shown in Table 3. For impacts at 24 km/hr (15 mph) and 15 degrees, a 30.5-cm (12-in.) mounting height successfully redirected the test vehicle for both the square and rectangular shapes with no tendency for vehicle climbing. However, for the same impact conditions, a 35.6-cm (14-in.) mounting height was required to successfully redirect the vehicle for the trapezoidal shape. The trapezoidal shape with a 30.5-cm (12-in.) mounting height allowed the tire to climb up and over the curb. These tests indicated that inconsistencies in the previous testing were not caused by paint applied to the trapezoidal rail but may have been a result of the drying creosote or the temperature changes mentioned previously. Following these tests, it was determined that one full-scale vehicle crash test would be performed on one of the successful curb shapes. The square shape with a 30.5-cm (12-in.) mounting height was selected for full-scale vehicle crash testing because it offered the most cost-effective design alternative.

TABLE 3 Summary of Curb-Type Bridge Railing Development, Phase II

Rail Type (cm x cm)	Rail Height (cm)	Test No.	Speed (km/h)	Results
Trapezoid -20.3 x 22.9	20.3	10a 10b 10c	24 24 24	Failed - vehicle over top of curb Failed - vehicle over top of curb Failed - vehicle over top of curb
Rectangle - 10.2 x 30.5	20.3	11	24	Failed - vehicle over top of curb
Rectangle - 10.2 x 30.5	25.4	12	24	Failed - vehicle over top of curb
Rectangle - 10.2 x 30.5	30.5	13a 13b	24 24	Passed - right-front tire briefly popped into air Passed - right-front tire briefly popped into air
Trapezoid - 20.3 x 22.9	30.5	14a 14b 14c	24 24 24	Passed - minor vehicle uplift action Passed - right-front tire climbed onto curb Failed - vehicle over top of curb
Trapezoid - 20.3 x 22.9	35.6	15a 15b	24 24	Passed - no climbing tendency Passed - no climbing tendency
Square - 20.3 x 20.3	35.6	16	24	Passed - no climbing tendency
Square - 20.3 x 20.3	30.5	17a 17b	24 24	Passed - no climbing tendency Passed - no climbing tendency

Full-Scale Crash Testing, Phase II

One full-scale crash test (LVCS-4) was conducted on the 20.3- by 20.3-cm (8- by 8-in.) square shape with a 30.5-cm (12-in.) mounting height attached to the longitudinal timber deck. In Test LVCS-4 the vehicle hit the curb rail at a speed of 23.2 km/hr (14.4 mph) and an angle of 15 degrees. Impact occurred approximately 3.35 m (11 ft) from the upstream end of the 11.9-m (39-ft) long installation, as shown in Figure 4. The square shape with a 30.5-cm (12-in.) mounting height successfully redirected the vehicle, which came to rest approximately 22.0 m (72 ft) downstream from the impact, as shown in Figure 4. A summary of the test results and the sequential photographs are presented in Figure 5.

Except for minor scuff marks on the right-side tires, there was no visible vehicle damage, as shown in Figure 4. No damage occurred to the curb rail or steel hardware. In addition, the glulam timber deck was not damaged.

The curb-type bridge rail contained and redirected the test vehicle without penetrating or overriding the bridge rail. Detached elements, fragments, or other debris from the bridge rail did not penetrate or show potential for penetrating the occupant compartment and would not present any hazard to other traffic or pedestrians. The integrity of the occupant compartment was maintained with no intrusion or deformation. The vehicle remained upright during and after collision, and the vehicle's trajectory did not intrude into adjacent traffic lanes. The vehicle exit angle of approximately 0 degrees was less than 60 percent of the impact angle or 9 degrees.

The curb bridge railing successfully redirected a 1999-kg (4,406-lb) pickup truck driven at a speed of 23.2 km/hr (14.4 mph) and an angle of 15 degrees. The curb bridge railing met the performance evaluation criteria (i.e., structural adequacy, occupant risk, and vehicle trajectory) for crash testing bridge railings (*1,2*) but at the reduced impact conditions of 24 km/hr (15 mph) and 15 degrees.

Breakaway Bridge Railing

Design Considerations

A steel W-beam railing with timber bridge posts was selected for use in the flexible bridge railing design based on previously crash-tested metal beam bridge railings (*12–14*), economics, and material availability. Breakaway posts rather than stiff posts were chosen in order to keep material costs below $33/m ($10/ft) by reducing the required structural capacity of the post-to-deck attachment. The post-to-deck attachment was designed so that no damage would occur to the timber deck or connection hardware. A side-mounted post-to-deck attachment with no rail or post blockouts was selected in order to reduce the required minimum width of timber deck.

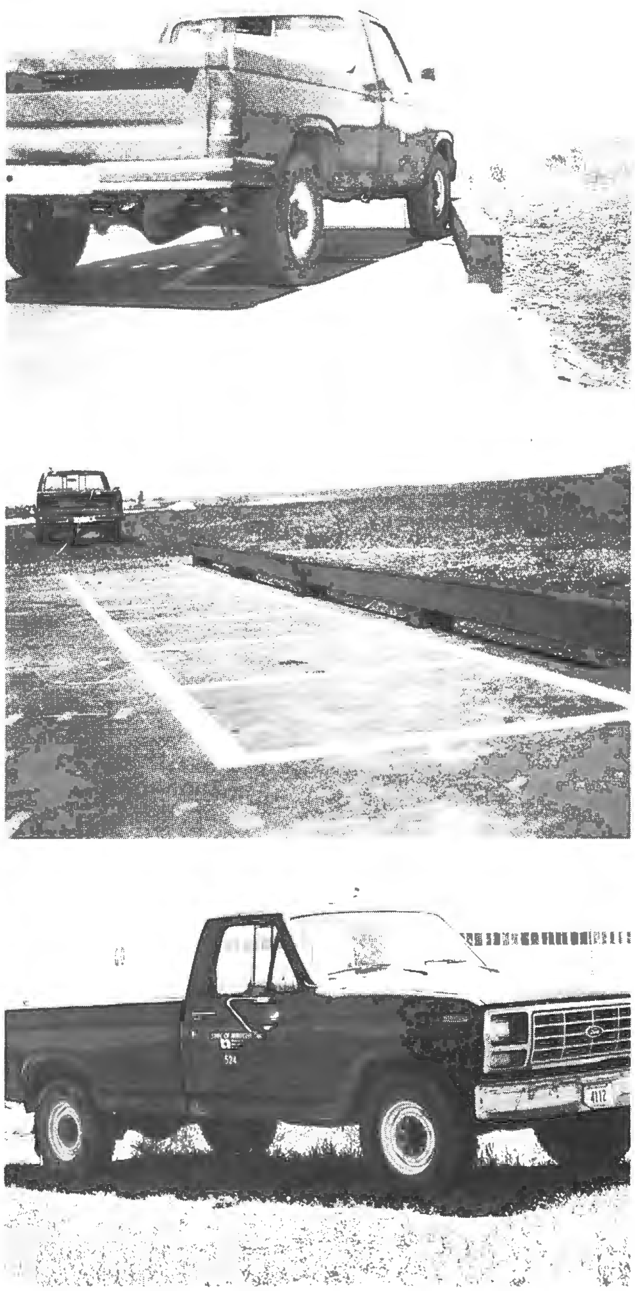


FIGURE 4 Impact location, vehicle trajectory, and vehicle damage, Test LVCS-4.

Static Post Testing

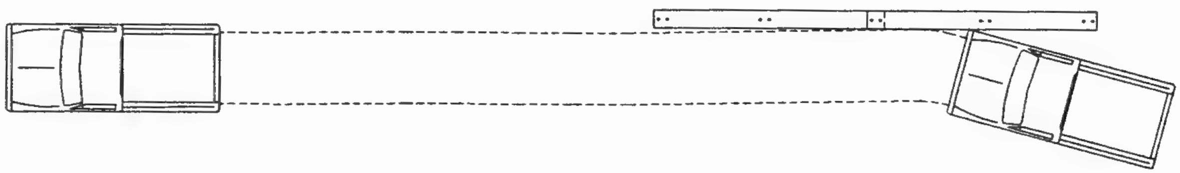
Static post testing was used to determine the force-deflection characteristics of two dimensions of lumber post sizes, 10.2-cm (4-in.) by 10.2-cm (4-in.) and 10.2-cm (4-in.) by 15.2-cm (6-in.) nominal. The cantilevered posts were bolted between two steel angles and attached to a rigid plate. Various angle sizes were used during the testing in order to determine the optimum angle dimensions. Thirteen static tests were performed. A 10.2-cm (4-in.) by 15.2-cm (6-in.) lumber post measuring 83.8 cm (33 in.) long with steel angles measuring 12.7 cm (5 in.) by 12.7 cm (5 in.) by 1.0 cm (3/8 in.) was selected for the original design. The maximum static force for this post size was 10.7 kN (2.4 kips). The post and angle sizes were selected based on a maximum force level that would not damage the post-to-deck attachment hardware or the deck.

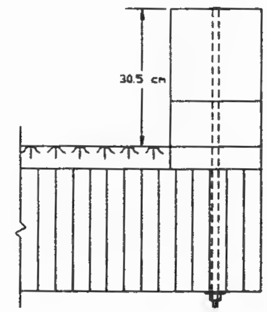
Following the failure of the first full-scale crash test, 24 additional static tests that included increasing the post height and placing saw cuts in the compression zone, tension zone, and combinations thereof were performed. A 10.2-cm (4-in.) by 15.2-cm (6-in.) lumber post measuring 93.3 cm (36.75 in.) long with steel angles measuring 12.7 cm (5 in.) by 12.7 cm (5 in.) by 1.0 cm (3/8 in.) was selected for the modified design. The modified posts also included a 2.5-cm (1-in.) horizontal saw cut placed on the tension side of the post 7.6 cm (3 in.) from the base of the post. The maximum static force for this post size was 5.8 kN (1.3 kips). Ritter et. al (*15*) provide additional details for the static post testing.

Design Details

A standard 12-gauge W-beam rail was selected for the rail element with a 61.0-cm (24-in.) top mounting height. However, after failure of the first full-scale crash test, the rail height was modified to 55.0 cm (21.65 in.) as measured from the top of the asphalt surface to the center of the rail. This provided a new rail top mounting height of approximately 70.6 cm (27.78 in.). In addition, the flat washer located under the head of the W-beam bolt was removed. The bridge rail was supported by 15 posts spaced on 1.90-m (6-ft 3-in.) centers. The chromated copper arsenate (CCA) treated lumber posts measured 10.2-cm (4-in.) by 15.2-cm (6-in.) nominal or 8.9-cm (3.5-in.) by 14.0-cm (5.5-in.) actual dressed size. The lumber posts were manufactured using Douglas fir Grade No. 2 or better. A 1.6-cm (5/8-in.) diameter by 17.8-cm (7-in.) long ASTM A307 galvanized hex head bolt attached the rail to each post. Each post was placed between two 12.7-cm (5-in.) by 12.7-cm (5-in.) by 1.0-cm (3/8-in.) by 15.2-cm (6-in.) long ASTM A36 galvanized steel angles. Two 1.6-cm (5/8-in.) diameter by 14.0-cm (5 1/2-in.) long ASTM A325 galvanized hex head bolts attached the post between the angles. Each post with attached angles was rigidly fixed to the outside vertical surface of the timber deck with two 1.9-cm (3/4-in.) diameter by 30.5-cm (12-in.) long ASTM A307 galvanized lag screws. A

0.000 sec | 0.064 sec | 0.275 sec | 0.437 sec | 1.229 sec





Test Number	LVCS-4
Date	5/12/93
Bridge Rail Installation	Low-Volume Curb Bridge Rail
Length	11.89 m
Timber Curb Rail	
Size	Square 20.3 cm x 20.3 cm
Top Mounting Height	30.5 cm
Material	Douglas Fir
Grade	No. 1
Preservative Treatment	Creosote
Timber Scupper Block	
Size	10.2 cm x 20.3 cm x 30.5 cm
Material	Douglas Fir
Grade	No. 1
Preservative Treatment	Creosote
Anchorage Bolts	
Type	ASTM A307, Galvanized
Size	Two 1.6-cm ϕ Bolts Per Location
Length	71.1 cm
Spacing	2.90-m Centers

Bridge Deck Installation	Longitudinal Glulam Timber Bridge Deck Panels
Panel Size	27.3 cm x 1.22 m x 5.72 m
Material	Glulam Timber Deck Comb. No. 2
Vehicle Model	1985 Ford F-250 Pickup
Test Inertial Mass	1,999 kg
Gross Static Mass	1,999 kg
Vehicle Speed	
Impact	23.2 km/h
Exit	Not Available
Vehicle Angle	
Impact	15 degrees
Exit	0 degrees
Vehicle Snagging	None
Vehicle Stability	Satisfactory
Maximum Vehicle Rebound Distance	Not Applicable
Bridge Rail Damage	None
Vehicle Damage	None
Vehicle Stopping Distance	21.95 m

FIGURE 5 Summary of test results and sequential photographs, Test LVCS-4.

schematic of the modified breakaway bridge railing is shown in Figure 6.

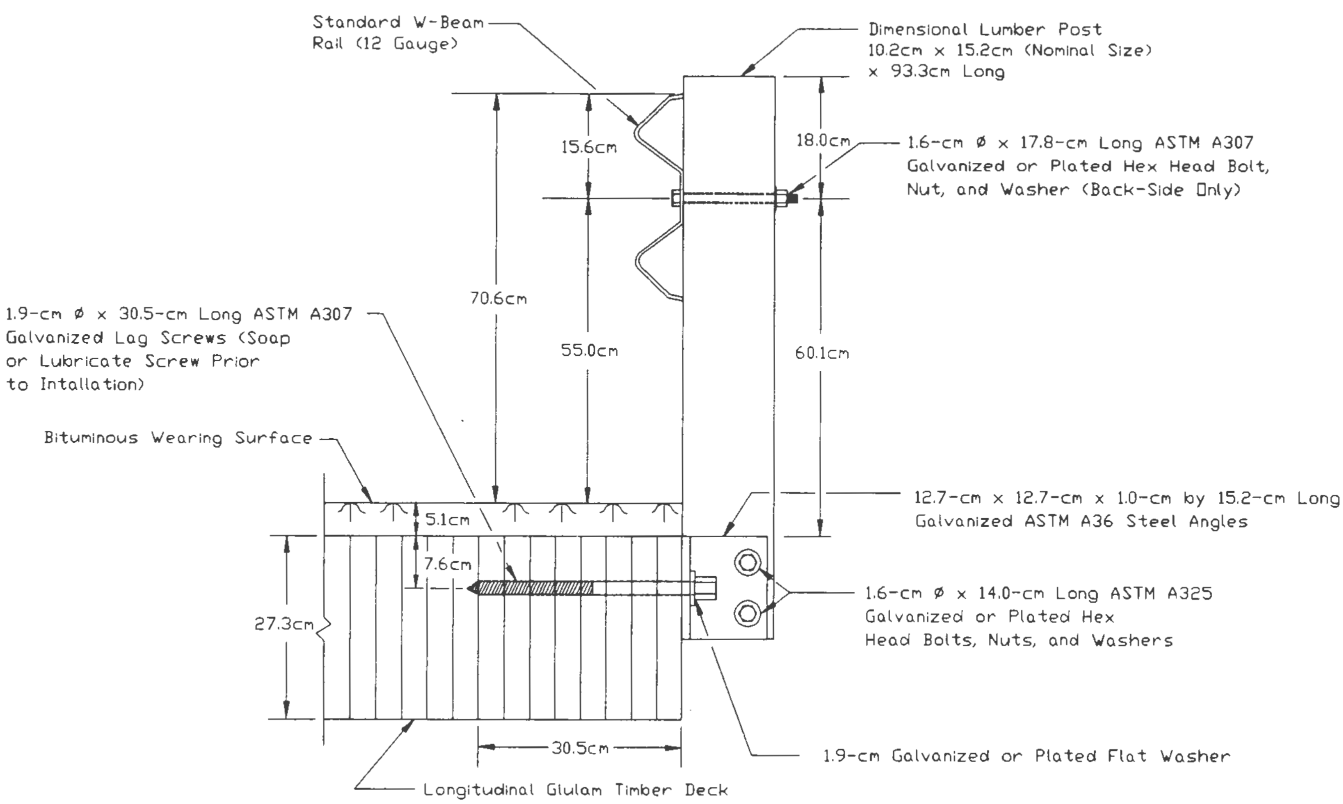
Approach guardrails were placed on each end of the bridge railing. The bridge railing with approach guardrails was 60.96 m (200 ft) long. Each W-beam approach guardrail was 15.24 m (50 ft long) and supported by 15.2-cm (6-in.) by 20.3-cm (8-in.) timber posts spaced on 1.90-m (6-ft 3-in.) centers. Guardrail anchorage was provided at each end by a modified breakaway cable terminal (MBCT) with steel foundation tubes, bearing plates, and channel struts.

The bridge railing was attached to a longitudinal glulam timber deck supported by concrete abutments. The concrete abutments, longitudinal glulam timber deck, and asphalt surface were the same as those used in the development of the curb-type systems.

BARRIER VII Computer Simulation Modeling

Following the preliminary design of the breakaway bridge railing, computer simulation modeling with BARRIER VII was performed to analyze the dynamic performance of the bridge railing before full-scale crash testing (*16*). Computer simulation was conducted modeling a 1996-kg (4,400-lb) pickup truck driven at 31 mph (500 km/hr) and an angle of 25 degrees according to Test Level 1 of NCHRP Report 350 (2).

The simulation results indicated that the original and modified breakaway bridge railing designs satisfactorily redirected the 1996-kg pickup truck. For the modified design, computer simulation predicted that eight breakaway lumber posts would be broken during impact, and the maximum permanent set and dynamic deflections



Notes:
(1) Post Spacing 1.90m
(2) Sawcut in post not shown.
(3) Many details have been omitted.
(4) 1in. = 2.54cm

FIGURE 6 Modified breakaway bridge railing.

of the W-beam were predicted to be 56.1 cm (22.1 in.) and 82.5 cm (32.5 in.), respectively. In addition, the predicted peak 0.050-sec average impact force perpendicular to the bridge railing was approximately 27 kN (6 kips).

Full-Scale Crash Testing

Two full-scale crash tests were performed with 3/4-ton pickup trucks on a breakaway bridge railing. The first test, LVBR-1, was conducted on a 61.0-cm (24-in.) high W-beam rail (original design), and the second test, LVBR-2, was conducted on a 70.6-cm (27.78-in.) high W-beam rail with a 2.5-cm (1-in.) saw cut on the tension side of the post (modified design). It was not necessary to conduct a full-scale crash test with a 820-kg minicompact hitting at 50 km/hr (31 mph) and 20 degrees since there was no potential for occupant risk problems arising from wheel snagging caused by the weak timber posts and low impact speed.

Test LVBR-1 (Original Design)

A 1984 Chevrolet C-20 pickup truck weighing 2041 kg (4,499 lb) struck the bridge rail at Post No. 7 at 50.2 km/hr (31.2 mph) and 26.8 degrees. Upon impact, the vehicle's bumper was forced over the top of the W-beam rail. The vehicle's tires then climbed up the face of the W-beam and the vehicle vaulted over the bridge rail. Failure of the bridge rail was attributed to insufficient rail mounting height. Damage to the connection angles and lag screws was also noticed.

Test LVBR-2 (Modified Design)

A 1985 Chevrolet C-20 pickup truck weighing 2043 kg (4,504 lb) struck the bridge rail at Post No. 7 at 49.2 km/hr (30.6 mph) and 24.9 degrees, as shown in Figure 7. A summary of the test results and the sequential photographs are shown in Figure 8. The vehicle became parallel to the bridge railing at 0.652 sec with a velocity of 38.8 km/hr (24.1 mph). Although the vehicle was redirected, it did not exit the bridge railing. The vehicle came to rest 13.4 m (44 ft) downstream from impact with the vehicle's left-side tires and right-side undercarriage resting on the deck surface, as shown in Figure 7. At no time, during impact or at any time thereafter did the vehicle's right-side tires contact the ground.

Vehicle damage was minor. Following the crash test, the vehicle's right-side tires were lifted onto the deck, and the vehicle was driven away. Damage on the right-front quarter panel was caused by vehicle-rail contact, and damage to the right-side undercarriage was caused by contact with the outer top surface of the deck, as shown in Figure 7. Bridge rail damage was also minimal, as shown in Figure 9. One 1.90-m (6-ft 3-in.) section of W-beam rail was permanently damaged. Eleven posts, Nos. 4 through 14, fractured away from the deck attachment. Five steel angles were deformed downstream of impact because of contact between the angles and the undercarriage of the vehicle.

The modified breakaway bridge rail contained and redirected the test vehicle without allowing it to penetrate or override the barrier. Detached elements, fragments, or other debris from the bridge rail did not penetrate or show potential for penetrating the occupant compartment and would not present any hazard to other traffic or pedestrians. The integrity of the occupant compartment was maintained with no intrusion or deformation. The vehicle remained upright during and after collision, and the vehicle's trajectory did not intrude into adjacent traffic lanes. Thus, the modified breakaway bridge railing successfully met all the evaluation criteria for Test Level 1 of NCHRP Report 350 (2).

Conclusions

Curb-Type Bridge Railing

The square-shaped bridge rail with a 30.5-cm (12-in) mounting height successfully redirected the pickup truck after an impact at a speed of 23.2 km/hr (14.4 mph) and an angle of 15 degrees. This result is consistent with the results from Phase II of the developmental testing program. Full-scale crash tests were not performed on the trapezoidal and rectangular shapes with 35.6-cm (14-in.) and 30.5-cm (12-in.) mounting heights, respectively. However, based on findings from the developmental testing program, it was reasoned that these shapes would behave similarly to the square-shaped curb rail and did not require additional full-scale crash testing.

Thus, three curb-type bridge railings were developed for longitudinal timber decks located on low-volume roads, as shown in Figure 10. The top-mounted timber curb railings provide economic and aesthetically pleasing bridge railing alternatives. Material costs for the three curb-type bridge railing systems are reasonably low. The rectangular-shaped railing system has the lowest material costs at $39.60/m ($12.07/ft), and the trapezoidal-shaped railing system has the highest material costs at $47.08/m ($14.35/ft). In addition, the curb-type railing system is easy to install and should have low construction labor costs. These railing systems could easily be adapted to other types of longitudinal timber decks. Finally, no bridge deck or railing damage was observed during testing on a longitudinal glulam deck system. Thus, maintenance and repair costs associated with the new curb designs should be very low.

Modified Breakaway Bridge Railing

A flexible railing with a breakaway post system was developed and successfully met the Test Level 1 crash test conditions of NCHRP Report 350 (2). The 70.6-cm (27.78-in.) high W-beam bridge rail successfully redirected a 3/4-ton pickup truck after an impact at 49.2 km/hr (30.6 mph) and an angle of 25 degrees. The side-mounted railing provides an economic railing with readily available materials. Material costs for the bridge railing system are reasonably low at approximately $25.85/m ($7.88/ft). In addition, the breakaway railing system is easy to install and should have low construction labor costs. This railing system should also be adaptable to other types of longitudinal timber decks. In addition, no bridge deck damage was observed after testing; therefore, repair costs should also be kept to an absolute minimum.

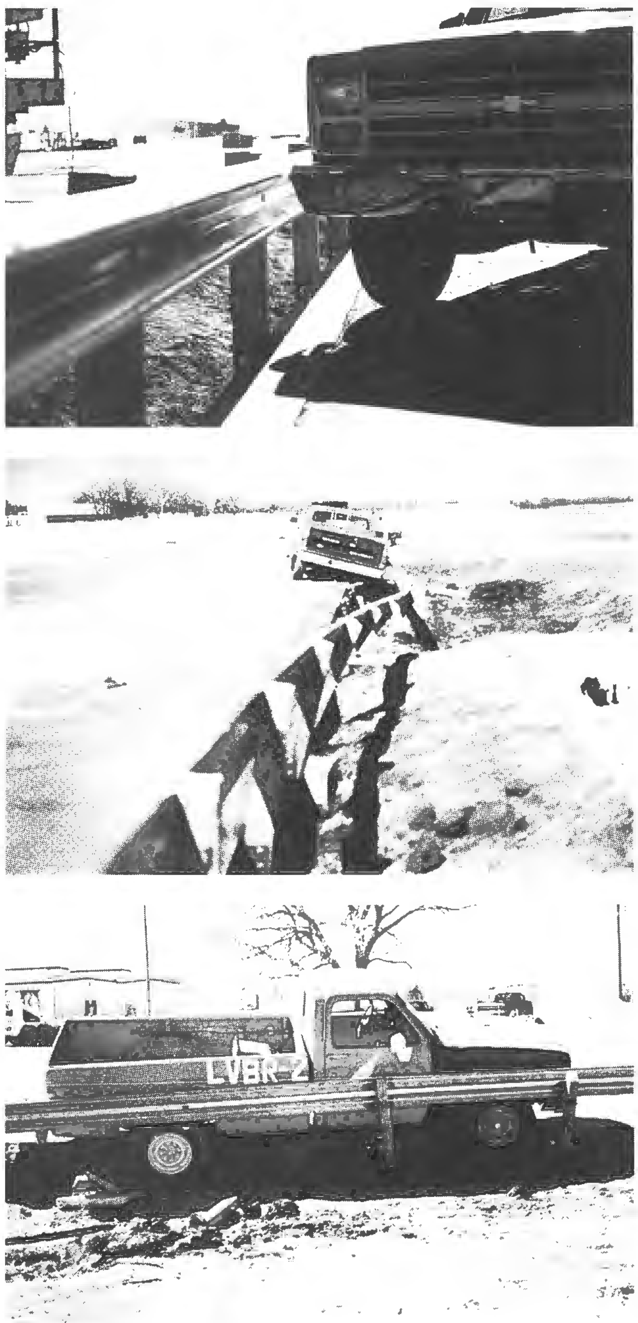


FIGURE 7 Impact location, vehicle trajectory, and vehicle damage, Test LVBR-2.

Discussion and Recommendations

The curb and breakaway bridge railings described herein were developed for low-impact condition requirements. The developmental testing program indicated that the redirective capacity of the curb railings could be increased by modifying the curb height and size, the rail-to-deck attachment, and the capacity of the rail splice connection. Curb railings should be able to meet the performance requirements of Test Levels 1 and 2 of NCHRP Report 350 (2). These higher-performance timber curb railings could be adapted for use in many different barrier applications. As bridge railings, the curbs would provide an aesthetic and economic alternative to conventional steel and concrete railings on many low-volume streets and highways with increased driver visibility. For flexible railings with breakaway posts, the full-scale crash testing program indicates that acceptable impact performance is possible although large dynamic rail deflections can be expected. Therefore, flexible railings with a modified post-to-deck attachment and stronger posts may be able to meet the performance requirements of Test Level 2 from NCHRP Report 350 (2).

Thus, it is recommended that the research described herein be extended to develop higher-performance timber curb railings and barriers and flexible railings for timber bridge decks.

ACKNOWLEDGMENTS

The authors would like to thank the following organizations for their contributions to the success of this research project: the American Institute of Timber Construction (AITC), Vancouver Washington, for donating the glulam materials for the deck construction and the Office of Sponsored Programs and the Center for Infra-

0.000 sec	0.316 sec	0.474 sec	0.652 sec	0.830 sec

Test Number	LVBR-2
Date	3/18/94
Bridge Rail Installation	Low-Volume Breakaway Bridge Rail
Length	30.48 m
Steel W-Beam Rail	
Size	12 Gauge
Top Mounting Height	70.6 cm
Posts (No. 1 through 15)	
Size	8.9 cm x 14.0 cm x 93.3 cm
Material	Dimension Lumber (CCA)
Grade	No. 2 or Better
Bridge Deck Installation	Longitudinal Glulam Timber Bridge Deck Panels
Panel Size	27.3 cm x 1.22 m x 5.72 m
Material	Glulam Timber Deck Comb. No. 2
Vehicle Model	1985 Chevrolet C-20 Pickup
Test Inertial Mass	2,043 kg
Gross Static Mass	2,043 kg
Vehicle Speed	
Impact	49.2 km/h
Exit	NA

Vehicle Angle	
Impact	24.9 degrees
Exit	NA
Vehicle Snagging	None
Vehicle Stability	Satisfactory
Effective Coefficient of Friction (μ)	0.28 (Fair)
Occupant Impact Velocity - normalized	
Longitudinal	2.2 m/s (9 m/s) (2)
Lateral	1.9 m/s (9 m/s) (2)
Occupant Ridedown Deceleration - 0.010-msec average	
Longitudinal	4.3 g's (15 g's) (2)
Lateral	3.8 g's (15 g's) (2)
Vehicle Damage	Minor
TAD	1-RFQ-1
VDI	01RFEW1
Maximum Vehicle Rebound Distance	Not Applicable
Bridge Rail Damage	Minor Rail Deformation and Eleven Fractured Posts
Maximum Dynamic Deflection	131.8 cm
Maximum Permanent Set Deflection	115.3 cm

FIGURE 8 **Summary of test results and sequential photographs, Test LVBR-2.**

FIGURE 9 Bridge rail damage, Test LVBR-2, three views.

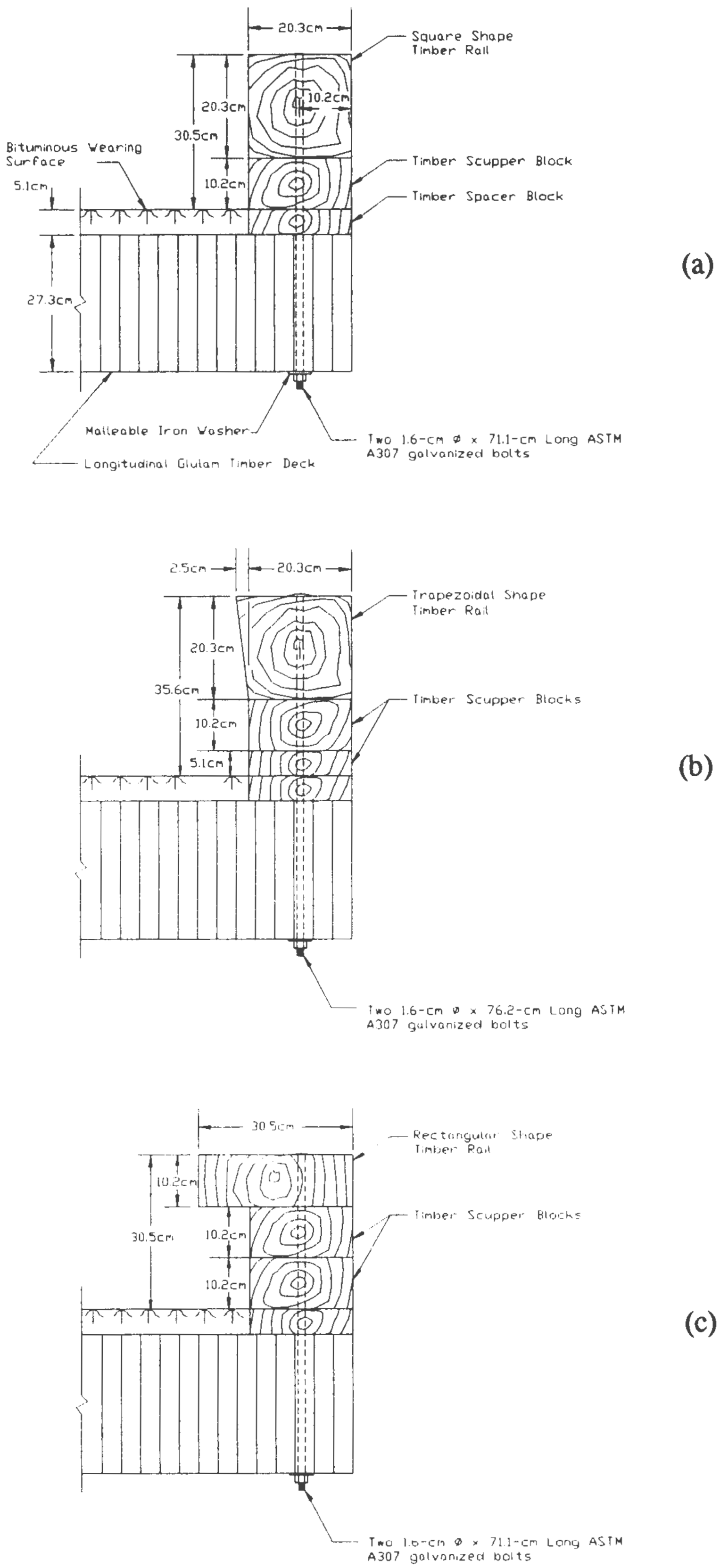


FIGURE 10 (*a*) Square-shaped curb, (*b*) trapezoidal-shaped curb, (*c*) rectangular-shaped curb.

structure Research, University of Nebraska-Lincoln, Lincoln, for matching support.

REFERENCES

1. *Guide Specifications for Bridge Railings.* American Association of State Highway and Transportation Officials, Washington, D.C., 1989
2. *NCHRP Report 350: Recommended Procedures for the Safety Performance Evaluation of Highway Features.* TRB, National Research Council, Washington, D.C., 1993.
3. *NCHRP Report 86: Tentative Service Requirements For Bridge Rail Systems.* TRB, National Research Council, Washington, D.C., 1970.
4. *Guide for Selecting, Locating, and Designing Traffic Barriers.* American Association of State Highway and Transportation Officials, Washington, D.C., 1977.
5. Ritter, M. A., R. K. Faller, and D. L. Sicking. *Development of Low-Volume Curb-Type Bridge Railings for Timber Bridge Decks.* Forest Products Laboratory, USDA Forest Service, Madison, Wisc., 1994.
6. Guidry, T. R., and W. L. Beason. Development of a Low-Profile Portable Concrete Barrier. In *Transportation Research Record 1367*, TRB, National Research Council, Washington, D.C., 1992.
7. Segal, D. J. *Highway-Vehicle-Object Simulation Model—1976*, 4 vols. Reports FHWA-RD-75-162, 163, 164, 165. FHWA, U.S. Department of Transportation, 1976.
8. *American Wood-Preservers' Association Book of Standards.* American Wood-Preservers' Association, Woodstock, Md., 1991.
9. Faller, R. K., M. A.. Ritter, J. C. Holloway, B. G. Pfeifer, and B. T. Rosson. Performance Level 1 Bridge Railings for Timber Decks. In *Transportation Research Record 1419*, TRB, National Research Council, Washington, D.C., 1993.
10. Ritter, M. A., R. K. Faller, J. C. Holloway, B. G. Pfeifer, and B. T. Rosson. *Development and Testing of Bridge Railings for Longitudinal Timber Decks by Full-Scale Crash Testing.* Forest Products Laboratory, USDA Forest Service, Madison, Wisc., 1994.
11. Ritter, M. A., R. K. Faller, and B. T. Rosson. *Development of the TBC-8000 Bridge Railing.* Forest Products Laboratory, USDA Forest Service, Madison, Wisc., 1994.
12. Stoughton, R. L., J. R. Stoker, I. Nagai, P. Hale., Jr., and R. W. Bishop. *Vehicle Impact Tests of a See-Through, Collapsing Ring, Structural Steel Tube, Bridge Barrier Railing.* Report FHWA/CA/TL-83/05. Office of Transportation Laboratory, California Department of Transportation, June 1983.
13. Hirsch, T. J., J. J. Panak, and C. E. Buth, *Tubular W-Beam Bridge Rail.* Report FHWATX78-230-1 or TTI-2-5-78-230-1. Texas Transportation Institute, Texas A&M University, Oct. 1978.
14. Bronstad, M. E., and J. D. Michie. *NCHRP Report 239: Multiple-Service-Level Highway Bridge Railing Selection Procedures.* TRB, National Research Council, Washington, D.C., Nov. 1981.
15. Ritter, M. A., R. K. Faller, and B. T Rosson. *Development of a Low-Volume Breakaway Bridge Railing for Timber Bridge Decks.* Forest Products Laboratory, USDA Forest Service, Madison, Wisc., in press.
16. Powell, G. H. *BARRIER VII: A Computer Program for Evaluation of Automobile Barrier Systems.* Report FHWA RD-73-51. FHWA, U.S. Department of Transportation, April 1973.

The **Transportation Research Board** is a unit of the National Research Council, which serves the National Academy of Sciences and the National Academy of Engineering. The Board's purpose is to stimulate research concerning the nature and performance of transportation systems, to disseminate the information produced by the research, and to encourage the application of appropriate research findings. The Board's program is carried out by more than 330 committees, task forces, and panels composed of more than 3,900 administrators, engineers, social scientists, attorneys, educators, and others concerned with tr... without compensation. The program is supported by state transportation ... U.S. Department of Transportation, an ... development of transportation.

The National Acade... of distinguished scholars engaged in sc... e of science and technology and to thei... ter granted to it by the Congress in 186... deral government on scientific and techi... onal Academy of Sciences.

The National Acad... r of the National Academy of Sciences, ... autonomous in its administration and in ... demy of Sciences the responsibility for ... Engineering also sponsors engineering ... ion and research, and recognizes the su... sident of the National Academy of En...

The Institute of M... Sciences to secure the services of emine... of policy matters pertaining to the heal... en to the National Academy of Sciences ... government and, upon its own initiati... n. Dr. Kenneth I. Shine is president of ...

The National Res... ciences in 1916 to associate the broad ... pose of furthering knowledge and advi... eneral policies determined by the Aca... both the National Academy of Science... ces to the government, the public, an... dministered jointly by both the Academ... Robert M. White are chairman and v...

Officials

AASHO	
AASHTC	
ASCE	
ASME	
ASTM	
FAA	
FHWA	Federal Highway Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ITE	Institute of Transportation Engineers
NCHRP	National Cooperative Highway Research Program
NCTRP	National Cooperative Transit Research and Development Program
NHTSA	National Highway Traffic Safety Administration
SAE	Society of Automotive Engineers
TRB	Transportation Research Board

NATIONAL ACADEMY PRESS

ISBN 0-309-06074-5