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Low-Volume Roads: Environmental Planning and Assessment, Modern Timber Bridges, and Other Issues

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Foreword

The 11 papers in this Record are presented in three parts. The four papers in Part 1 present information on international efforts in the area of environmental planning and assessment. Adverse environmental impacts associated with low-volume roads can be prevented or greatly reduced through careful planning and the use of appropriate assessment procedures. Construction and maintenance of low-volume roads, which provide the first link in the roadway networks, are essential for the development of agricultural and natural resources and for making basic social services accessible to rural communities. But the construction, maintenance, and operation of low-volume roads also can have negative environmental impacts, such as soil erosion and landslides; water pollution from surface runoff contaminated with residual fuels, oil, herbicides, and dust palliatives; degradation and siltation of streams; interference with natural drainage, flooding, and damage to wetlands; air pollution from dust blown by vehicles; visual pollution; and alteration of land use patterns. In some cases the socioeconomic and cultural impacts of low-volume roads, particularly in remote areas, may be even more significant than environmental impacts. Socioeconomic and cultural impacts may include the spread of contagious diseases and pests, spontaneous migration and invasion of tribal lands, disruption of indigenous communities and their traditional lifestyles, uncontrolled logging, conversion of forest to pastureland, massive degradation of marginal lands, destruction of wildlife habitats, and the introduction of exotic species of flora and fauna.

Jansson presents the environmental impact assessment practice in Finland. For rural road schemes, formalized assessment methods and reporting may not be appropriate or cost-effective. Greater emphasis should be placed on the assessment process, including familiarization with local conditions, extensive cooperation with affected local residents and authorities, and an enlightened approach to rural road planning to integrate environmental factors with road location and design decisions. Moll reports on the basic principles of road location and drainage as well as good maintenance practices for minimizing adverse environmental impacts. Vásárhelyi presents the Hungarian road transport situation and its related environmental impacts. Standards for road design and operation in Hungary will be supplemented by a set of "principles for the environment of roads," applicable to low-volume roads. Wolfe discusses the use of geotextile-reinforced bituminous seals, an emerging highway technology, to improve amenity and environment of remote rural roads.

The five papers in Part 2 provide the results of laboratory or field tests or analysis of various modern timber bridges. Ritter et al. describe the construction and performance monitoring of a stress-laminated deck bridge. Testing included moisture content measurements, bar force levels, and deflection response to live loads. Buttlar and Mozingo address the testing of a two-layer timber deck system. According to the authors, by using two layers of wood, a stressed timber bridge can be constructed of a less expensive and more widely available lumber. Davalos et al. describe the design, fabrication, erection, and testing of a modular stress-laminated T-system timber bridge. Manbeck et al. describe a case history of a hardwood glued-laminated bridge and give an overview of the development of the hardwood glued-laminating process. Barger et al. report on their laboratory testing of Tee and Box beam models to evaluate stresses and deflections resulting from various loadings.

The two papers in Part 3 discuss other issues related to low-volume roads. Greenstein provides information on issues related to administration of low-volume roads in developing countries. Sacco et al. describe a device that they developed for measuring the cross-sectional deformation of aggregate-surfaced roadways.
PART 1

Environmental Planning and Assessment
Environmental Impact Assessment and Evaluation of Low-Volume Roads in Finland

ANDERS H. H. JANSSON

Environmental impact assessment (EIA) practice in Finnish road design is as yet an informal procedure; a law on EIA is expected in 1993. Methodology in development since 1990 has been tested on several projects, most of them large scale. Low-volume rural roads are ideal for introducing many of the objectives and procedures of EIA, but formalized schemes in such small projects should be avoided. The Finnish experience shows that the attitude of the designer, willingness to accept as valid points of view not one's own, thorough familiarity with local conditions and goals, and extensive cooperation with local people and authorities are essential to the success of the EIA as well as to the design of these roads. A process view on EIA emphasizes the tasks related to project initialization and start-up. If a process of active cooperation characterizes the whole project, formal reporting and decision stages will have a minor importance.

A Finnish law on environmental impact assessment (EIA) is to be adopted in 1993 (1), as the formation of the European Economic Area brings the European Community directive on EIA into force in Finland. The proposed law is modeled on the 1991 United Nations Economic Commission for Europe convention on the assessment of transboundary effects (2). The Finnish National Road Administration has been developing an EIA methodology since 1990 on the basis of international experience and the proposed law. The first Road Administration guideline on EIA was published in 1992 (3).

Regulations on assessment concern only large-scale projects. However, in low-volume rural road projects an integrated and participatory process where environmental concerns are given an equal footing with technical and other goals can be easily realized.

Development efforts aim to set standards for both the legally required EIA and the "local EIA" so that they will strengthen each other by interaction, large-scale EIA introducing the formal system and local EIA familiarizing road designers and others with the attitudes and methods of environmentally integrated cooperation.

THE RURAL ROAD, A BACKBONE OF DEVELOPMENT

After World War II, Finland had to replace the resources consumed by the war, resettle the inhabitants of areas ceded to the Soviet Union, and develop an agricultural society into a modern, urban, and industrial one. The Finnish road network, inadequate in the early 1930s, was in a catastrophic condition by the late 1940s. Large areas were without functional road connections. An enormous road-building effort centered on renewing the main roads network was necessary. But the rapid extension of the smaller rural roads was a simultaneous process. Without these roads, resettlement or development in rural areas would not have been possible.

Today, out of 77 000 km of public roads, connecting and collector roads form 57 000 km and carry 30 percent of total public road vehicle mileage. The average daily traffic on connecting roads is 280 vehicles and on collector roads 650. Fifty percent of the public roads have gravel surfacing and 30 percent light bituminous surfacing (4). These roads are still the backbone of regional structure. Though 80 percent of our population live in urban areas, an efficient rural road network is the basis of continued rural habitation.

FINNISH ROAD DESIGN PROCEDURE

Design of public roads is regulated by the Road Act. At present, the Road Act specifies only the legal approval of the final design. In addition, the act provides for preliminary design stages, but without specifying the procedures to be followed.

In practice, the road design procedure has developed into a system of plans, programs, and preliminary and final design stages (5). Project design starts with Road Administration feasibility or preliminary location studies. For projects deemed feasible and necessary, a project decision is made. Depending on project size, the decision is made by the Regional or Central Road Administration or the Transport Ministry. The decision does not specify alternatives to be studied but sets general goals for the project.

Road projects are designed by the Regional Road Administrations, assisted by consultants. The preliminary design aims at fixing the main features of the project, including its general alignment. The goals are interpreted into specific objectives. Alternatives are studied and assessed. From an environmental point of view, this is the most important design stage. It ends with the project decision, giving the Regional Road Administration the right to commence final design. An approved project decision is a condition for including the project in the road construction program.

The final design stage was originally the only stage of road design. As new stages have been added, it has taken on a technical character, implementing the main features decided...
upon at the preliminary design stage. The final design is the basis for legal approval of the project. The approval decision is made by the Transport Ministry. Landowners and authorities can appeal against this decision.

A proposal for a new Road Act is being prepared. The preliminary design stages are specified in the proposal. The project decision and its legal consequences would be included in the act. This would allow appeal against the project decision, thus extending public influence on road design (6).

EIA LEGISLATION AND ROAD DESIGN

The main steps of the assessment procedure in the Law on Environmental Impact Assessment, as proposed by an Environment Ministry task force (1), are as follows (see Table 1 and Figure 1):

- Deciding whether a project is subject to assessment,
- Making an assessment schedule and consulting on the schedule,
- Performing the assessment,
- Publishing the assessment document and consultation, and
- Deciding on the project, taking the assessment into account.

For road projects, the Regional Board of Administration would be the authority directing the process. The board would have the right to appeal against project approval if it finds that the EIA has not been properly carried out.

Appended to the law is a decree on implementation. The projects subject to the law are listed in the decree. For roads, they are motorways and semimotorways. The law contains criteria for determining the environmental significance of projects not specifically listed:

(1) The location of the project and its possible effects on important natural and cultural values and objects of special sensitivity as well as human health, living conditions and welfare,
(2) the extent of the area of influence of the project, and
(3) the possible combined effects of the project and other projects and activities. (I, translation by the author)

These criteria extend the law to main and major trunk road projects as well as most projects in dense urban areas.

The Road Administration guideline requires an EIA for all projects that have at least some significant impacts. The guideline groups projects as follows:

- Projects with significant environmental impacts over large areas (motorways, semimotorways, and projects deemed to

<table>
<thead>
<tr>
<th>TABLE 1 Proposed EIA Legislation in Finland (1)</th>
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<tr>
<td>ONE EIA LAW, PROJECT DECISIONS BY SEPARATE LAWS</td>
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<td>ASSESSMENT OF PROJECTS, PLANS AND PROGRAMS</td>
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<td>POSITIVE LIST AND CRITERIA OF SIGNIFICANCE</td>
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<td>COORDINATION WITH OTHER PROCEDURES</td>
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<td>EARLY ASSESSMENT</td>
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<td>COMPETENT AUTHORITY DIRECTION</td>
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</table>
Collector and connecting roads usually have local impacts. They are subject to "local EIA" development. A preliminary design stage is not required, and the procedures in use for regional and large-scale projects are not appropriate.

ENVIRONMENTAL CONCERNS OF LOW-VOLUME RURAL ROAD DESIGN

Low-volume rural road design is traditionally based on minimizing construction outlays and maximizing local utility, for instance by providing direct access to as many habitations as possible. Essentially, these considerations tend to support environmentally responsible design. A close fit to geological and hydrological features, an economy of terrain limiting cuts and embankments, consideration of landownership and boundaries, and service of a specific local traffic need will all contribute to fitting the design to landscape and land use.

On the other hand, minimizing design expenditure may result in insufficient inventories or alternative studies. Maximizing utility may impair traffic safety as well as objects or areas of environmental importance. A short alignment may destroy village structure or the landscape. Thus, a main environmental concern of low-volume rural road design is to ensure a general alignment supportive of landscape and land use. The alignment should also, if possible, avoid intruding into nature reserves or other areas of importance for the preservation of nature, since land use development will follow roads.

On a technical level, environmental concerns are directed toward keeping the footprint of the road as narrow as possible, protecting groundwater catchment areas and other natural resources, and effectively utilizing existing road alignments. In a landscape with numerous lakes and rivers, crossings form a special problem. Bridges and embankments cannot be avoided, but their technical requirements presuppose a scale out of keeping with the road itself.

PUBLIC ROAD 5053

In 1991, extension of EIA to projects with local impacts was tested on Public Road 5053, in eastern Finland. This road, with an average daily traffic of 240 to 350 vehicles, serves the Eno municipality and connects a paper plant in Uimaharju village and the Koli nature park area with the main road network. There was a need to improve the technical standard and the alignment of the road. The total length of the project is about 20 km. The major impacts would be caused by changes to 3 km within the village of Ahveninen and 5 km within the Paukkajanvaara-Kaltimonlahti forest area, to the west of the village.

The road design procedure was in its final stages when the North Karelia Regional Road Administration (NKRA), in cooperation with the Regional Board of Waters and the Environment (NKBWE), decided to perform an environmental assessment. During the summer of 1991, NKBWE made a study of the features of the Paukkajanvaara-Kaltimonlahti area. NKRA concentrated on impacts on Ahveninen village.

A report on the assessment was prepared by NKRA in cooperation with the Central Road Administration (7). The

For projects subject to the law, the assessment procedure would be included in the preliminary design process, which the present Road Act does not recognize.

Thus legal EIA requirements are directed toward approval of the final design. From an environmental viewpoint, this is too late, since all the crucial decisions were made earlier. Until the Road Act is revised, the assessment document will be appended to both the preliminary and the final design.

For projects with regional impacts, typically minor trunk road and major collector road projects, the Road Administration guideline recommends the same general procedure as for large-scale projects.

For projects with environmental impacts on a regional scale: These projects are assessed in accordance with the Road Act.

For projects with local environmental impacts: These projects are assessed by Regional Road Administration decision in a way that realizes the principles of assessment. The administration decides on how the assessment is performed.

For projects with few or insignificant environmental impacts: No assessment procedure is foreseen. Environmental impacts that may arise are to be documented and, if need be, mitigated (3, translation by the author).

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report noted that the assessment process was started too late to have significant effects on road design. Although the village inhabitants believed that such late consultation would not significantly influence decision making, they appreciated that NKRA took the initiative to explain and discuss the project and its impacts. Two village meetings were held and a series of interviews performed. Villagers proposed several alternatives to the projected alignment. New alternatives were included in the assessment, as were the “no action” alternative and a “do minimum” alternative that would not change road alignment.

On the basis of the discussions held, the assessment took the form of a description of the impacts of the alternatives on the village. The factors are given in Table 2. For the forest area, similar alternatives were formulated and assessed by NKBWE. The 4-month assessment timetable was too short for adequate coordination between NKRA and NKBWE.

As a result of the assessment, NKRA made minor changes in its original proposal. However, the questions raised by this process have changed the system of interagency cooperation in the region as well as NKRA programming practices. A follow-up report, extending the conclusions of this and other similar cases, was prepared by the Central Road Administration (8).

**EIA PARADOX**

The low-volume rural road has a paradoxical role when seen in the EIA context. It is not in the public interest to add a formal EIA methodology to the design of low-volume road projects. Legal and formal requirements should be restricted to projects that can have severe impacts over large areas. But the relatively small scale and continuous flow of projects make low-volume roads ideal for realizing the substance of EIA. Conflicts of interest are fairly easy to identify, alternatives are limited, and local solutions possible. A single road designer can, with the assistance of “visiting experts,” manage an entire project through continuous communication and cooperation with the local people. No multilevel planning organization is needed.

But this somewhat rosy picture does not really bridge the gap between the road engineer and the environment the project will change. Conflicts arise and remain unresolved. Popular sentiment may see road building as a steamroller with little concern for local values:

> The original Ahveninen villagers feel cheated. They were told that the Road Administration had introduced a new procedure, giving local people a say in road design. They did have their say: there was one who came to each household asking questions and meetings were held, but . . . [7, quoting Karjalanen newspaper (translation by the author)]

EIA methodology development has been tested on several other projects. Conclusions are to a great extent centered on matters of attitude independent of scale. Indeed, specific methods of data gathering, prediction, or assessment need development mostly in large-scale projects; in low-volume road projects their importance is secondary. A low-volume road EIA process stands on the following:

- The professional competence of the road designer,
- The interests and environmental objectives arising out of the locality itself, and
- The values of the local inhabitants and their willingness to actively cooperate throughout the design process.

**TABLE 2 Public Road 5053, Assessed Factors (7)**

<table>
<thead>
<tr>
<th>1. FACTORS ASSESSED FOR AHVENINEN VILLAGE</th>
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<tbody>
<tr>
<td><strong>TRAFFIC SAFETY</strong> To ensure the best possible traffic safety for goods transport, passenger car traffic, pedestrians and cyclists.</td>
</tr>
<tr>
<td><strong>TRAFFIC SERVICE LEVEL</strong> To ensure an optimal traffic service level for goods transport and cars.</td>
</tr>
<tr>
<td><strong>VILLAGE LANDSCAPE</strong> To preserve the village landscape, especially the open field areas and the characteristic village road.</td>
</tr>
<tr>
<td><strong>VILLAGE STRUCTURE</strong> 1. To preserve village structure, alternatively 2. To change village structure in order to enhance the possibilities for new service outlets.</td>
</tr>
<tr>
<td><strong>NATURAL VALUES</strong> To minimize ecological damage.</td>
</tr>
<tr>
<td><strong>AGRICULTURE</strong> To minimize field area loss and severance of farm areas.</td>
</tr>
<tr>
<td><strong>CONSTRUCTION AND MAINTENANCE</strong> To ensure an optimal technical standard and costs of construction and maintenance.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>2. FACTORS ASSESSED FOR THE PAUKKAJANVAARA-KALTIMONLAHTI FOREST AREA</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>ECOLOGY</strong> To minimize ecological damage.</td>
</tr>
<tr>
<td><strong>NATURAL LANDSCAPE</strong> To minimize disruption of landscape elements and to minimize visual intrusion.</td>
</tr>
<tr>
<td><strong>SENSITIVE AREAS AND OBJECTS</strong> To preserve areas and objects of special ecological sensitivity or importance.</td>
</tr>
</tbody>
</table>
PROFESSIONAL COMPETENCE AND ENVIRONMENTAL COMPETENCE

The road designer's competence is mainly technical. There is a readiness to avoid damaging objects or areas seen as important, but the need to improve the designer's competence is illustrated by the question, "What is important?" Here, the decision given by the designer will often differ from one given by environmental authorities and others.

The problem is not that the designer does not know how to identify a rare bird. One can always ask. But one must be willing to ask and to accept an answer by somebody competent in another field than one's own as relevant. To achieve this, the designer needs a broad view: to see areas and processes as a whole and to realize how the identification of a valuable feature can define an important area. Even a vague notion, received sufficiently early, is an indication that can influence a whole project.

Vision is difficult to transmit by teaching or marketing methods. At present we encourage the Regional Road Administrations to organize seminars with local universities on environmental themes. Such seminars, with the local footing given by the university and representatives of local authorities, may start a thought process with practical consequences. It is a question of prying attitudes loose, freeing the ordinary thought of the designer from the strictures set upon it by earlier training and organizational uniformity. If that succeeds, the engineer's bent will drive most designers to find new visions and tools by themselves.

Thus, the first step would be to use the road designer's competence and attempt to change the attitudes, then let the designer's own ambition widen that competence.

UNDERSTANDING THE LOCALITY

Everybody understands one's own neighborhood or town and how different influences and processes have shaped its development. But in a professional context, the understanding is clouded by real or assumed standards or practices. On low-volume road projects, clouding is most easily seen in, for instance, straightening a curve to some standard radius even though no accidents or other impairments have occurred, or widening a carriageway from 7 to 8 m. A standard value is followed, whether or not it is needed locally.

Some standards are obligatory; the road network may not vary indefinitely. But the application of these standards must never be automatic. Every decision must be appropriate for the place and needs concerned.

Even on small roads, some large-scale interests may be present. The transport needs of heavy industry or occasional tourist rush traffic are among these. In Finland, the most common are national environmental protection interests. These may or may not coincide with local interests. But their presence carries the problem to another level: national interest implies a full-scale EIA procedure and its methods of solving conflicts.

On low-volume roads, local interests and environmental goals are generally predominant. The designer should be made aware of their role and should withhold judgment until attaining an understanding of what this very place is about.

Technical and construction decisions should never be made before determining that they are necessary.

Thus, the second step is to demand that the designer be thoroughly familiar with the locality, its inhabitants, and its terrain before drafting any alternatives. No technically motivated solution should be accepted unless a convincing need is shown.

COOPERATION

On low-volume road projects, the conditions for cooperation between designers and the public are ideal. The concerned population seldom exceeds 1,000; its active representatives are perhaps a few dozen people. One can meet face to face, discuss on the spot, and show the proposed road by putting markers into the ground or walking along a line.

Traditionally, the engineer, as a representative of state power and officialdom, has been seen as above ordinary people. The chairman of the local authority was invited to meetings, others not, nor did they presume to intrude. This tradition is by no means dead. Communication based on such a tradition cannot result in active or continuous cooperation.

A starting point is asking people what is important. Then, one can present possible ways of reaching the important goals by this one method, road construction, and let people evaluate whether it is worth it. The back-and-forth will continue through the stages of formulation and evaluation of alternatives, proposals for action and mitigation, and so on. This is also the natural way of presenting a report of proceedings to people. What is then put into the formal report is of minor importance.

The third step is making a conscious effort to contact the concerned people and keeping them involved in the real design and decision making. Then, one can expect continuous cooperation. Presentations "for discussion" of decisions already made are seen through and mean the end of cooperation.

PROCESS VIEW OF EIA

EIA development for low-volume rural roads encourages a process view of assessment. The basis of EIA is the need for an environmentally responsible design process. The procedural constraints set by law are simply a framework to give the process an administratively efficient shape. On low-volume rural road projects, formal stages of design are few. The assessment process shows itself as a natural way of organizing the designer's work.

A description of the process from the designer's viewpoint could take the following form (Figure 2):

Project Initiation

The technical motivation for a road project may arise out of accident black spots, damaged road structure, difficulties for pedestrian or bicycle traffic, or a notable rise in traffic volume. The motivation of regional development exists if a road is substandard as a connection in the road network or if it generates unnecessary traffic because it does not serve land use effectively. An environmental motivation can be that the road...
it should serve. A designer is named and a preliminary timetable and budget are set. The decision is published.

The designer’s first task is to shape the framework of the project. Views and preconceptions are tested by visiting appropriate authorities and by a “walking conference,” informing people beforehand where and when the designer will be studying the terrain. At this stage, all available material, from maps and aerial pictures to local authority plans and inventories, is gathered. It is an advantage if this collection and at least part of the designer’s work can be located in the area concerned.

These studies should lead to a preliminary set of alternatives, including “no action” and “do minimum.” The number of alternatives can be large, each alternative serving some specific goal. Avoidance of ready-made compromise alternatives helps determine whether there are real conflicts in goals or values. A sufficiently large number of alternatives helps to clarify which possible impacts or targets are important for this project.

Environmental and other experts are called upon to determine what kind of further research and inventory is needed and how long they will take. Even in these projects, some biological field studies could take a couple of years.

The full schedule of research, assessment, and design of the project should be published, not just a separate EIA schedule. Preliminary goals and alternatives are reported, giving for each the constraints and relationships known and what studies will be made.

**First Assessment**

The aim of the first assessment stage, which runs parallel to terrain inventories and other studies, is to narrow the alternatives, especially to discard those having grave deficiencies. A balancing of technical and environmental criteria will take place. This presupposes a new round of discussions. At this stage, it is to be expected that local opinion has to some extent solidified and group interests—and, conversely, defined interest groups—show up. An interest group can be an authority, the landowners of a village, a local history society, and so forth.

If possible, a committee of participating experts and interest group representatives is formed to carry on discussions. This is a mutual learning stage, each teaching the others to analyze the problem from its own viewpoint. The road designer need not be in charge. That position could be held by a local representative. Inventory data and study results are brought into the process continuously.

Consensus is sought, but it may not emerge. Two or three main alternatives are formed to express any conflicts as clearly as possible. The alternatives are presented to the public and views are invited. The views of authorities can be given as preliminary statements. In presentation, the emphasis is on general readability and a review of uncertainties or difficulties rather than on technically finished design material.

**Final Assessment**

The object of the final assessment is a choice of one alternative or a decision to drop the project. The previous discussions disturb housing, impairs landscape, or may constitute a threat to valuable objects or areas.

If the Regional Road Administration has seen that a need to improve a road or build a new one may exist, or a public initiative to that effect has been made, the matter should be discussed in a wide context. The administration can call people to a hearing or invite them to give their views in other ways. At this stage, no project need be specified. Discussions are based on the present road network and the problems it may contain; the goals for development of the area, and of its environmental qualities, that people and various authorities have; and the Road Administration’s scope of action: what can it do and what are, in general, the effects of a road project.

The last point implies another sector of environmental assessment, which we call “product and service assessment” (9).

It is based on the idea of technology assessment. In this respect it means that, for instance, a class of roads, or construction activities, should be studied in general as a basis for predicting what to demand or expect from a specific project. This type of assessment would assist in simplifying studies of small-scale projects.

**Start-Up**

On the basis of the discussions held, the administration decides whether there are grounds for starting a project and what goals

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**FIGURE 2 Process of design and assessment of a low-volume rural road project.**

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form the basis for deciding what features of each alternative should be studied further.

Experts have an important role in this stage. Formal prediction methods may be used. The interest group committee acts as a sounding board, forcing the experts to clarify their professional knowledge in nontechnical language. The committee aids the road designer in decisions on priorities.

Impacts on future land use and social concerns will need special study. The secondary and tertiary land use adjoining a small road has much greater impacts than the road itself. Notable social impacts can be brought about because the road, though small, may be the strongest development factor in an outlying village. Usually, development plans concerning the immediate vicinity of the projected road are sketchy or nonexistent.

In this respect, the interest group committee is a primary source for the researcher. The interests represented are mainly the ones that shape development. The local authority may at this stage take on the task of drawing up zoning or other plans, but discussions within the committee will also give a fairly reliable picture of what development trends may be expected and of how people value the effects of the project on their own future and welfare.

In this stage, technical precision will also be needed. Recommendations on alternatives must be technically feasible and costs must be calculated. In tight spots, exact drawings may be needed, though they should not be used as the main design tool. A design report is drawn up following the general contents of an EIA report.

**Process Decision**

The report is presented to the public, possibly in a second walking conference. The responsibility of deciding which alternative to choose rests with the Regional Road Administration, but the process should by now have given a clear indication of how to choose. The decision states what alternative is chosen, on what grounds, and how this alternative has been formed by the design process. Questions still to be solved through detail studies are identified, as well as any detrimental effects the alternative may have and how they can be mitigated.

**Final Design and Formal Decision**

The design is finalized by the road designer, checking open details with the concerned people. At this stage, the drafting and map style of engineers' design is used. The formal decision on the road project is the approval of the final design, as defined by the Road Act. For this, official statements are given by the authorities. The design is shown publicly. Landowners and others can present complaints against it, if necessary.

**CONCLUSION**

In a low-volume rural road project, formal procedure should be avoided. The experience and attitude of the road designer are paramount in devising an environmentally responsible design process. A spirit of open inquiry should prevail. Specific technical solutions and expressions should, in the main, be limited to the final stage of design.

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Reducing Low-Volume Road Impacts on the Environment: Success in the United States Department of Agriculture Forest Service

JEFFRY E. MOLL

Principles for low-impact roads have been under development since the 1960s. Observation of roads after storms and floods showed minimal damage occurring to well-located roads, with less environmental impacts in adjacent areas. Roads should have rolling grades and should be located to be self-maintaining and to "lay lightly" on the land. These roads minimize modification of existing drainage patterns. A sufficient number of properly located and designed cross drains should be provided. Location in the best spot is not always possible, however, and roads-related erosion, failures, and environmental damage can still occur. Also, road work may be planned and executed to promote healing of the scars of the past. Raising cross-drain pipe inlets encourages water ponding, soil conservation, the healing of gullies, and increased soil moisture. Road fills may serve as gully plugs or may be constructed with porus characteristics to reduce water concentration on broad, relatively flat meadows. Finally, a "less is better" approach to road maintenance may aid in sediment reduction. Providing for adequate drainage with the minimum disturbance during ditch pulling, heeling, and surface blading activities also has economic benefits. Tailoring ditch geometries to actual drainage needs, as well as compacting and arming the soil in the ditch, can reduce sediment levels. Flavel bars are useful in breaking up the concentration of water.

The United States Department of Agriculture (USDA) Forest Service, organized in 1905, administers a network consisting of approximately 552,000 km (343,000 mi) of road, of which 75 percent is functionally classified as local, 20 percent collector, and 5 percent arterial (1). The Forest Service is considered one of the foremost authorities on low-volume roads in the free world, mainly due to the large mileage of locals managed. Local roads typically have little or no surfacing and serve relatively low traffic volumes.

USDA Forest Service employees are striving to reduce the agency's transportation system impacts on the environment. The material presented in this paper is based mainly on the reflections of Forest Service engineers on some of their experiences. The paper presents material on low-impact roads, the healing of environmental scars of the past through road work, and reduction of sediment through road maintenance.

A common theme to be derived from the experiences of the engineers is the importance of careful study and planning before beginning any transportation system activity. Performance of tasks in accordance with textbook methods and attitudes such as "that's the way we've always done it" are giving way to an awareness of environmental concerns. In addition, it has been realized that all factors must be carefully weighed to obtain optimal solutions from engineering, economic, and environmental standpoints.

PRINCIPLES OF LOW-IMPACT ROADS

By the 1960s in the Pacific Northwest Region, road engineers and locators began to take note of low-impact roads, roads that weathered storms and floods while others suffered severe damage and even failure. Location was a common denominator among these roads; minimum impact on the environment was achieved by well-located roads. It was realized that location is the single most important phase of transportation system development (B. Sawyer, unpublished data). Forest Service experience in the Pacific Northwest Region allows for formulation of general principles of low-impact roads, and they appear valid for other geographical areas as well.

Heavy rains and floods in the early 1960s damaged roads and caused failures that highlighted several roads-related problems:

- Roads concentrate water and alter the natural drainage patterns that exist on and below the surface of undisturbed topography.
- Road surfaces resist infiltration, leading to increased runoff that can saturate fills and contribute to slides, slumps, and washouts.
- Road ditches collect surface water and intercept subsurface flow, concentrating and depositing it as runoff in spots not always well suited to handle it.
- Roadway cuts and fills disturb vast areas of soil, exposing it to weathering action and the elements. Road construction accelerates surface erosion more than any other forest management activity (2). Protective cover on soil is removed and the natural hillside is replaced by an artificial "flat" that provides the traveled way. This flat is compensated for by steeper-than-natural cut and fill slopes.
- Controlling the movement of water on slopes presents unique difficulties.

Two basic principles for low-impact roads emerged from careful study of different road segments and their weathering of...
storms and floods. The first is that of proper location. No amount of extra effort in design work or construction technique makes up for a poor location, but a good location facilitates subsequent road development activities. Inspection after heavy storms reveals survival of well-located segments and, generally, failure of or damage to those in poor locations.

The proficient road locator possesses considerable experience in transportation system development activities and must develop an in-depth knowledge of the characteristics of the area in which the road is to be located. Proper location requires the locator to have expertise in the following areas: transportation planning, including road design elements and standards and design criteria; road surveying, design, cost estimating, construction, and maintenance; aerial photograph interpretation and the use of contour maps; drainage structure design for surface and subsurface flow; materials engineering and soil mechanics; and application of newly emerging construction materials and techniques.

In addition, the locator should have the following knowledge of the area: types of land use planned, including land and property ownership; natural resources (also cultural, visual, and recreational resources and any associated constraints); the existing transportation system; geomorphologic, topographical, and geological characteristics; and hydrology and climate.

The second principle of low-impact roads is locating and designing a road to minimize the alteration of existing drainage patterns (B. Sawyer, unpublished data). Almost any modification of the natural drainage process results in water concentration and increased erosion potential.

- Ridgetop roads require less provision for drainage than side hill or canyon bottom roads. Roads close to streams face greater risk of road-related sediment entering the stream.
- Care must be taken to ensure proper cross-drain location and design and that adequate drains are provided to minimize water concentration and other impacts on the hydrology of the area (R. A. LaFaette, unpublished data).
- Pipe outlets should be designed to prevent damage to fills, erosive soils, meadows, and streams and to encourage the spreading of outflow.
- Full bench construction alleviates the problems associated with saturated fills but still modifies natural slope and hydrologic characteristics and can actually intensify the interception of groundwater. The "toe of the cut" is made further into the hill, since all required road width is provided by the cut and none by the fill.
- A southern aspect provides for faster melting of snow and quicker drying of road surfacing and subgrade.

Other important considerations for low-impact roads are as follows:

- Roads should be located on rolling rather than straight or uniform grades. A roll in the grade constitutes a dip that facilitates the shedding of water, helping to prevent water concentrations.
- Roads should be located, designed, and built to be as self-maintaining as possible. To be self-maintaining, a road resists runoff concentration, erosion damage, and vehicular damage such as surface rutting. Observation of road survival on a case-by-case basis suggests that self-maintaining roads inflict the least damage on the environment.
- A carefully thought-out road location that allows a road to "lay lightly" on the land by minimizing cuts and fills and other disturbed areas can reduce the area affected and other environmental impacts.
- Roads should be located on as gentle a side slope as possible, although some side slope facilitates drainage.
- Drainage of water from the road surface is more easily provided for on flatter grades; thus water concentration and erosion potential are reduced.
- Wetlands, bogs, and areas experiencing exfiltration of groundwater should be avoided during road location. These areas require costly mitigation and result in increased potential for environmental damage.

Other useful considerations that are not directly related to road location are as follows:

- Tailor road design standards to actual needs. Building the minimum standard road required to meet resource and access needs can have economic as well as environmental benefits.
- Steepen cut and fill slopes to reduce the total area affected and the disturbed area exposed to weathering and erosion.
- Optimize construction equipment to the job to prevent unintended overbuilding, extra road width, and excessive disturbance of adjacent areas. Hydraulic excavators allow for increased control over placement of excavated material, with the possibility of a more stable end product (3).
- Plan and conduct road construction activities to minimize disturbances (4).
- Plan and implement erosion control measures during and after construction. A seeding plan that includes a specially designed seed mixture to ensure reestablishment of vegetation before the rainy season is important.
- Provision for surface drainage should be carefully planned through the use of outslope, inslope, crown sections, and berms to control the flow of water off the road (see Figure 1).

It is not possible to locate every segment of every road perfectly. Difficult road-building situations will present themselves, and decisions will have to be made between alternatives that result in undesirable end products from environmental, economic, or engineering standpoints.

Road locators and designers are not always able to foresee or provide for all repercussions resulting from road construction activities. Whereas careful location, design, and construction of roads will minimize adverse effects, not all road-related erosion is preventable, and environmental damage and failures will occur (5). It is important to approach transportation facility development with these facts in mind.

**HEALING THE ENVIRONMENTAL SCARS OF THE PAST THROUGH ROAD WORK**

In the Southwestern Region, an increasing awareness of the environmentally detrimental aspects of transportation system-related activities and practices has resulted in a multidisciplinary approach to healing the scars of the past. Resource specialists initially recognized the contributions of roads to
man-caused damage inflicted on riparian areas, meadows, drainages, vegetation regimes, and wildlife habitat. Hence, investigations into its causes and possible cures were initiated.

Personnel in related disciplines at the Cibola National Forest, Albuquerque, New Mexico, are working together to identify areas damaged by human activities and devise ways to promote healing through road-related activities. Recognizing the damage done and identifying its causes and consequences are the first steps to be taken. Careful consideration of all aspects of the problem often leads to innovative solutions.

Experimentation shows what works well and indicates procedures that require modification.

Meadow invasion by ponderosa pine, juniper, rabbit brush, and other mesic vegetation, along with reduced hydric biomass, indicates the drying of soil and lowering of water tables (F. Jackson, unpublished data). By considering road cross-drain pipes installed with inlet elevations below the natural meadow surface, part of the reason for the damaged environment becomes clear. The practice of lowering a pipe inlet effectively removes water from the roadway but can initiate gully headcutting; relieving the pipe outlet usually requires cutting long trenches, also initiating gully erosion and concentrating water. Both practices result in dryer soil and a lowered water table, with disastrous results to wetland areas and hydric vegetation.

Raising the inlet of a cross-drain pipe, either by installing an elbow on the pipe inlet or by removal and reinstallation of the pipe at a higher elevation, circumvents its soil moisture draining and water table lowering characteristics (J. Fehr, unpublished data). The pipe inlet is generally raised to the natural meadow elevation; thus, runoff ponding occurs on the uphill side of the pipe, encouraging sedimentation and slowing headwall retreat. Downstream treatments include splash aprons and gully plugs. Aprons are used to reduce the force of falling water on soil and retard gully deepening and downcutting, whereas plugs encourage ponding and soil preservation.

Roadway embankments across drainages may be specifically designed to function as gully plugs (J. Fehr, unpublished data). Drainage structures are sized to pass the peak flow associated with the design recurrence interval required for the facility in question. The structure is placed so that its invert is at the elevation desired for water ponding and sediment deposition levels (Figure 2) rather than in accordance with the standard installation procedure of bedding the pipe at the natural ground elevation in the drainage bottom. Generally, the road surface should be at least 0.6 m (2 ft) above the water level to prevent increased moisture from decreasing the road's bearing capacity. Embankments are specifically designed and constructed to provide an impervious dam to prevent piping and possible loss of the fill, although in some cases large, uniformly sized rock is used to provide a pervious layer that allows seepage. The purposes of such embankments include wetlands creation and restoration; providing wildlife drinkers; the retardation of bank cutting, gully erosion, and incision; and sediment preservation.

Roadway embankments across broad, relatively flat meadows may be constructed in a manner that reduces the concentration of water (Figure 3). The fill is designed and constructed with a pervious layer as described above, allowing for seepage of surface water and sheet flow in a manner imitating the natural drainage pattern. Water need not be collected for passage through a pipe; it remains spread across a relatively large area, encouraging infiltration and soil moisture recharge. This application works best in broad, flat meadows without defined low spots or canyons that concentrate water.

Gully plugs may be constructed of Jersey barriers (Figure 4), treated timber, or on-site materials. Placing the plugs at carefully arranged intervals optimizes their sediment-preserving capabilities and economic benefits. A grade of approximately plus 2 percent from the top of one plug to the toe of the next one upstream has been successfully used. Installing plugs in conjunction with increasing the infiltration capacity of surrounding upland soils increases both plug survivability and the soil's biomass production potential.
Use of one culvert concentrates flow, causing downstream gully. Culvert bottom set below grade drains meadow and causes gully headcut migration.

French drain allows normal dispersed flows to seep through road fill. Drain, subgrade, and surface must be designed to carry normal loadings.

FIGURE 3 Improper and proper meadow crossings.

REDUCING SEDIMENT THROUGH ROAD MAINTENANCE

By 1961, Intermountain Region engineers had recognized the importance of road maintenance in the fight against erosion and sedimentation. Basic concepts had surfaced, including “keep the soil where it is” and “build maintenance into the road” (J. J. Wise, unpublished data). The problems inherent in managing many kilometers of road built to low standards—some on poor locations, with steep grades, little or no surfacing, and a lack of drainage provision—had been realized. Also, the realization that no two roads and no two erosion problems are exactly alike made it clear that instructions for equipment operators could not be written to cover all situations and that much would be left to the judgment of the on-site maintenance supervisor and equipment crew. It became apparent that each problem area should be analyzed separately before determining the proper corrective action for the situation and that operators should be trained in erosion control measures and effective equipment operation. During road maintenance activities, it is important to take into consideration all factors relevant to the situation rather than to routinely perform the maintenance. Familiarity with forest roads and the terrain, soil and vegetation, climate, and roadway characteristics provides the basis for making sound decisions that will allow for adequate drainage, safe travel, and resource protection.

(The material presented in the rest of this section is a condensation of a pamphlet by J. Firth.)

During ditch pulling activities, a “less is better” approach may sometimes be the best policy for sediment reduction. Selectively pulling ditches where required rather than routinely pulling all ditches has economic and environmental benefits. Lessening the amount of disturbance to the soil and vegetation in the ditch while still providing for adequate drainage will minimize sedimentation. During maintenance, stable cut slopes can be undermined, possibly causing slopes steeper than the material’s natural angle of repose, with resulting slides and slope failures contributing to blockages and sedimentation. Tailoring ditch geometries to actual drainage needs can also alleviate slope undercutting in oversize ditches or in ditches with small flow volumes.

Ditch heeling, which consists of storing accumulated ditch waste against the cut slope toe during successive operations until the amount of material bermed requires removal, is an alternative to ditch pulling. Heeling the ditch is effective when dealing with smaller material volumes and can result in savings in maintenance time and money while providing for adequate drainage and resource protection. The heeled material aids in cut slope stabilization and helps prevent material moving down the cut slope from reaching the ditch bottom, where it subsequently would be transported downstream.

Areas in which large volumes of material move from the cut slope into the ditch may require special treatments rather than simple removal and disposal of the material. When realignment of the ditch away from the slope is such that slope stabilization is not feasible, drainage may be provided through the slough material by placing two logs side by side but with a gap, then bridging the gap with a third log. Sloughing material covers the logs, while a limited amount of drainage can occur between the logs.

Compaction of soil in the ditch, with resulting reduced erosion potential, may be obtained by tilting the mold board forward during ditch pulling and heeling operations and also by running the grader tires in the ditch and on the foreslope. Accumulated rock may be spread out in the ditch bottom as armoring material to combat scour. Gully plugs constructed in steep ditches slow the flow, reduce its erosive potential, and encourage the deposition of sediment. Grass and scattered rocks left in the ditch also slow flowing water.

The decision to leave or place obstructions in a ditch must be carefully weighed so that they remain assets and do not become liabilities. Impeded drainage may cause flow over the road or standing water in the ditch, both of which represent ditch malfunction and possible safety and operations problems.

Culvert inlet basins often present what appear to be “man-made slide areas” due to the toe of the cut being made farther...
into the hill to provide room for the catch basin. As the pipe inlet and catch basin are cleaned, the slope above may be undermined, causing more material to slide downslope. Culvert extensions have been successfully used to raise the inlet elevation, allowing material stabilizing the toe-of-slope to remain while providing for flow into the pipe. The new flowline is maintained at an elevation low enough to prevent subgrade saturation and drainage bypassing the culvert. A ditch dam may be placed downstream of the inlet to retard flow-by.

Settlement basins are dug to provide a pool for deposition of sediment before entry of ditch water into a live stream. Not intended for large volumes of water, they are relatively small and lined with rock to prevent erosion.

One primary objective in road surface maintenance is to provide for drainage of surface water. Poor drainage leads to saturation of road surfacing and base, erosion, and sedimentation. Maintenance of the surface section (be it outsloped, insloped, or crowned) and of surface smoothness is necessary to proper surface drainage. Surface cross drains are useful in removing water from steeper sections of road and those prone to surface rutting. Breaking up the concentration of water is especially important in these situations. Surface cross drains include open-top culverts, intercepting dips, cross ditches, and flavel bars. Flavel bars are a cross between interceptors and water bars, consisting of a shallow V-shaped ditch that runs at approximately a 20-degree angle to the road centerline for a length of 15 to 23 m (50 to 75 ft). The berm is placed on the downhill side, and the water may be directed to either side of the road, depending on surface section. Flavel bars are used mainly on roads with limited traffic volumes and low vehicle speeds.

Preventive measures are sometimes required to limit erosion on fill slopes. Riprap at culvert outlets armors the ground, retards the formation of gullies, and allows water to disperse. Filter windrows composed of logging slash and vegetative material, when placed on fill slopes, are effective at passing water but preserving sediment. Perhaps the most effective erosion preventive is seeding. Straw, mulch, or erosion control blankets may aid in protecting highly erodible areas until the seed germinates and takes root.

CONCLUSIONS

Many success stories may be found in the annals of the USDA Forest Service concerning the efforts of its employees in reducing transportation system–related impacts on the environment. Roads are being planned, located, built, and maintained with environmental concerns as a top priority. Engineers have found that, with careful planning, road work may be executed in a way that minimizes adverse effects while still providing the required access to an area.

REFERENCES

Low-Volume Rural Roads and the Environment: Some Ideas and Experiences from Hungary

BOLDIZSÁR VASÁRHELYI

After a brief description of the Hungarian road network, aspects of accessibility, land use, fuel consumption, and emission of pollutants are discussed concerning low-volume paved and unpaved (dirt) roads. The old truth, “we have to pay for the road whether we build it or not,” is also valid for low-volume rural roads. Hence, if the paving of such a road is economically viable, subject to proceeding with the required care for the environment, the environmental aspects will also support the decision to pave it.

Hungary is in the central part of the Carpathian basin. Most of its territory (93 000 km²) is plain and hilly. The highest peak is approximately 1000 m above sea level.

The climate is temperate, with warm summers and sometimes cold and snowy, sometimes milder and rainy winters. (Average yearly temperature is 10°C, and precipitation is 550 to 700 mm/year.) The greatest part of the land is cultivated (70 percent), there are forests on about 18 percent, and the rest is uncultivated or used for urban, industrial, and other purposes.

GENERAL ENVIRONMENTAL ASPECTS IN THE DESIGN, CONSTRUCTION, AND OPERATION OF ROADS IN HUNGARY

The Hungarian road network consists of national and local public roads and private roads. The network’s main characteristics are given in Table 1.

The domestic vehicle fleet of Hungary in 1990 consisted of about 2,000,000 passenger cars, 240,000 trucks, 126,000 buses, and 400,000 motorcycles. In 1990 approximately 88 percent of passenger and 25 percent of goods transportation was by road, which includes local public transit, buses, and private vehicles. Thus the importance of roads in the Hungarian transportation system can be assessed. They are not as important as in the United States or Western Europe, but from an environmental standpoint the greater share of the railways and of public transport is clearly an advantage. Yet, the roads and road transportation have their environmental and social problems in Hungary, and there is a growing and sometimes fashionably exaggerated social sensitivity toward them.

These problems are essentially similar to those of all European countries and are aggravated by regional peculiarities:

- Excessive fuel consumption and pollutant emission of the socialist-made part of the vehicle fleet;
- Lack of funds for investment, modernization, maintenance, and operation of the transportation system; and
- An overall decline of morals, causing a tragic safety situation, waste of human and natural resources, lack of care for the environment, and so forth.

In recognition of the significance of environmental issues, in the last decades a great amount of normative legislative activity got under way. The first major item was the Parliamentary Act II of 1976 “on the protection of the human environment.” A separate ministry was created to officially address environmental and land use issues.

With regard to road transportation, several governmental and ministerial decrees, standards, and regulations have been passed. The recent technical regulations on motorway and highway design and operation will be supplemented by a set of “principles for the environment of roads.” The first draft came out in 1992 (J).

Environmental impact analysis is performed for all motorway and major highway construction and modernization projects. The tendency is to follow and duly implement the regulations of the European Community—as in all aspects of economic life.

Environmental issues have always been taken into consideration in the design and construction of transportation infrastructure in Hungary. With growing public concern for the environment, directives were elaborated for all transport branches (2).

The overall environmental problems related to motorized road traffic are similar everywhere. They must not be treated as an isolated issue but should be integrated into the construction, maintenance, and operation technologies of transportation systems (2).

Road transportation influences the environment in several ways. According to recent Hungarian guidelines and practice, the principal efforts to mitigate these effects are as follows:

- Air pollution: the influencing of modal split (preference of other modes, combined transport, and public transport); reduction of passenger car trips; improvements in motor vehicle fleets, vehicle operation, maintenance, traffic conditions, and vehicle fuels; and reduction of pollution caused by road construction (e.g., asphalt mixing plants);
- Noise: improvements in motor vehicle fleets; use of silent pavements and noise barriers on road construction and re-
TABLE 1 Hungarian Road Network, 1990

<table>
<thead>
<tr>
<th>Road Category</th>
<th>Paved (1000 km)</th>
<th>Unpaved (dirt) (1000 km)</th>
<th>Total (1000 km)</th>
<th>Veh/day (1000)</th>
</tr>
</thead>
<tbody>
<tr>
<td>National</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Main roads</td>
<td>6.7</td>
<td>-</td>
<td>6.7</td>
<td>7394</td>
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<td>Secondary roads</td>
<td>22.5</td>
<td>0.4</td>
<td>22.9</td>
<td>1710</td>
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<tr>
<td>Total</td>
<td>29.2</td>
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<tr>
<td>Local</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>In built-up areas</td>
<td>22.5</td>
<td>24.1</td>
<td>46.6</td>
<td>2410*</td>
</tr>
<tr>
<td>In rural areas</td>
<td>1.6</td>
<td>27.5</td>
<td>29.1</td>
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<tr>
<td>Total</td>
<td>24.1</td>
<td>51.6</td>
<td>75.7</td>
<td>n.a</td>
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<tr>
<td>Private</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Agricultural</td>
<td>13.5</td>
<td>33.7</td>
<td>47.2</td>
<td>n.a</td>
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<td>Forestry</td>
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<td>2.3</td>
<td>4.8</td>
<td>n.a</td>
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<tr>
<td>Total</td>
<td>16.0</td>
<td>36.0</td>
<td>52.0</td>
<td>n.a</td>
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<tr>
<td>Grand total</td>
<td>69.3</td>
<td>88.0</td>
<td>157.3</td>
<td>n.a</td>
</tr>
</tbody>
</table>

* In 1985, on a sample of 1700 km of paved roads.

Source: Közutak főbb adatai 1990 január 1 OKF, Budapest, 1990
(Principal Data of Public Roads of Hungary)

habilitation; systematic control and maintenance of vehicles; traffic engineering measures (ensuring smooth traffic flow); and reduction of noise emission of transportation establishments (e.g., commercial vehicle plants and repair shops);

- Waste materials (hazardous wastes produced in road transportation amount to 0.7 to 1 percent of the national total): treatment and depositing of slurries from vehicles washing installations; clean burning of used oils and oily wastes; collection and recycling of batteries and acidic wastes; and treatment of paint material wastes (a national network of temporary and final storage sites for hazardous wastes is under construction);

- Pollution of soil along the roads: reduction of quantity of deicing salts; substitution of NaCl by other materials; and elimination of lead from vehicle fuels; and

- Pollution of water: reduction of water use in vehicle maintenance plants; treatment and cleaning of wastewater in maintenance plants; and catching and sedimentation of water flowing off the roads.

LOW-VOLUME RURAL ROADS IN HUNGARY

Table 1 shows that some 100 000 km (i.e., two-thirds of all roads) are low-volume roads, and most are unpaved (dirt roads). These roads do not have a great national or regional importance but are significant for their vicinity. They often serve as the last access elements to areas with less than 200 inhabitants. Moreover, in several cases they represent the missing links in the local and even secondary national network. Approximately 39 percent of Hungarians live in villages with population under 10,000. With 10.4 million inhabitants, Hungary has a population density of 108 persons/km², much less than that of Western Europe but much higher than that of North America. Therefore, low-volume roads are important for most inhabitants of villages with a population under 2,000 and for many inhabitants of villages where 2,000 to 10,000 people live (16.6 and 22.2 percent of the population of Hungary, respectively). Most such roads are essential accesses to the surrounding agricultural lands and forests and allow crops, timber, and other material to be hauled efficiently and in good condition.

If low-volume rural roads are paved and maintained adequately, they do not create a hazard to their environment. Vehicles traveling along them present their effects individually. The design hourly volume is about 40 vehicles per hour (i.e., less than 1 vehicle per minute) in the peak hour. Environmental problems can easily be caused by improperly operated individual (e.g., overweight) vehicles.

Environmental issues are, even in spite of the low traffic volumes, important in the case of unpaved (dirt) roads. In Hungarian soil and climatic conditions, such roads are usually (a) dry and dusty (summer, early autumn); (b) wet, muddy, slippery, and even impassable for a long time after rainfall (spring, late autumn, winter); (c) hard-frozen (some weeks in winter); or (d) covered by snow after storms and impassable for days because of drifts.

Generally, their surface is very uneven, causing excess fuel consumption, increased emission of pollutants and noise, deterioration of the vehicle and its load, and extra costs and time losses for the road users. Often, such dirt roads occupy a very wide strip of terrain, because the drivers try to find more even surfaces to travel on.

Of course we must never forget that all-weather accessibility of living and working places, hospitals, schools, and so forth is a basic human right, independent of a domicile in Budapest or in a hamlet.
FUEL CONSUMPTION ON DIRT ROADS

To assess the detrimental effects, in June and July 1992 several fuel consumption measurements were performed on dirt roads in a sandy valley in Pest County (Domonyvolgy-Erdõkertes). It is graded every month, so major unevenness cannot develop on it.

The surface of this road varies according to the composition of the soil and variations of the microclimate (on sunscorched and shadowy, moist sections, etc.). Thus, in dry weather, it has some sections that are almost as hard as a paved road, whereas others make the vehicle sink 1 to 2 cm into the sand, and some other sections with loose sand cause vehicles to "plow" through the sand to a depth of 4 to 5 cm. Consequently, the rolling resistance varies considerably.

The first case corresponds to driving on a level road, the second to a gradient of 8 to 10 percent, and the third to a gradient of 55 to 65 percent. The attainable speed decreases on the worse surfaces because only lower gears can be used and the dynamic effects must be kept at a reasonable level. A "critical speed" can be defined as the optimum safe speed under the given road and vehicle conditions.

The effect on fuel consumption is shown in Figure 1 and Table 2 for a Romanian-made five-speed Dacia 1310 TLX passenger car (license of Renault). The figure shows absolute values of the fuel consumption and the critical speed. The measurements were made with an Ono-Sokki type fuel consumption meter. For each curve in the diagram, three to five points were measured with 10-km/hr speed intervals, and for each point at least eight measuring runs were made with flying start. The pressure in the tires corresponded to the value prescribed by the factory for solid pavements.

Table 2 presents the fuel consumption ratio. These results are for moist sand. Loose, dry sand has less resistance, and the vehicles whirr up much dust.

The very wet sands and soils (nearly flowing mud) behave like melting snow. The wheels push it before themselves, making a slush of 1 to 2 cm to a continuous hindrance of 4 to 5 cm. Under such conditions, the fuel consumption ratio can reach 3.5 to 4.0 compared with the asphalt road.

Measurements on other roads with other passenger cars and trucks gave similar results. For trucks, the load on the vehicle is also an important factor. Often, trucks cannot travel along a deteriorated, wet, or muddy dirt road with full loads.

Excess vehicle travel means increased fuel consumption and, consequently, increased emission of pollutants.

SOME PRACTICAL CONCLUSIONS AND A CASE STUDY

The paving of several such roads would be necessary. This means essentially construction of a new road but using the existing right-of-way. In Hungary during the preparation of the plans, designs, and the legal procedure for obtaining permission to construct the roads by the local urban authorities, environmental issues are being taken into consideration.

<table>
<thead>
<tr>
<th>Sinking into the Road (cm)</th>
<th>Gear</th>
<th>IV</th>
<th>III</th>
<th>II</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 (asphalt)</td>
<td>1.00</td>
<td>1.03</td>
<td>1.22</td>
<td>1.94</td>
</tr>
<tr>
<td>0 (dirt)</td>
<td>1.02</td>
<td>1.05</td>
<td>1.24</td>
<td>1.96</td>
</tr>
<tr>
<td>1-2 (dirt)</td>
<td>*</td>
<td>1.32</td>
<td>1.58</td>
<td>2.19</td>
</tr>
<tr>
<td>4-5 (dirt)</td>
<td>*</td>
<td>*</td>
<td>1.85</td>
<td>2.94</td>
</tr>
</tbody>
</table>

* It was impossible to travel in this gear.

Usually, the low traffic and the existing dirt roads do not pose a grave problem. Care is taken to avoid environmentally sensitive areas, such as well-defined and legally protected natural reserves, and to provide appropriate protective measures.

Systematic nationwide plans for closing the gaps in the secondary national road network are now being elaborated. For each of the 19 counties, these plans contain a map showing protected areas and the planned roads. The construction of the Lébeny-Tarnokrété connection in northwest Hungary (County Győr-Sopron), a major link of 7.8 km (3.3 km strengthening and 4.5 km paving of a dirt road) on the secondary network provides a case study.

During the design phase, after the construction permit was obtained, the newly paved section bordered the Hanság National Park, an important wetland. However, on December 1, 1990, the area of the Hanság National Park was extended to include the section to be built. A strong campaign was launched by some pressure groups against the road and against the interests of the local population. The issue went before the Commission for Environment of the Parliament, too.

Since the Hanság area was extended after the issue of a construction permit and route relocation would have been impracticable, in August 1991 a formal agreement between the Highway Directorate in Győr and the Directorate of the National Park was signed. Its principal points were as follows:

- The road will be built (it is now in operation).
- Along the stretch across the national park’s area, covering bushes will be planted on both sides to protect animals (bustards, etc.).
- For reptiles (vipers, etc.) and amphibians, 10 subterranean passages will be built.
- The new road will have a speed limit of 60 km/hr, stopping will be prohibited, animal warning signs will be posted, and no transiting heavy traffic will be allowed.
- The disturbance of the environment will be minimal during the construction and operation of the road.
- There will be an agreement between the signatory parties on the use of financial resources.

The cost of the special measures for environmental issues was about 5 million HUF from a total of 75 million HUF (about $1 million U.S.) in construction costs.

CONCLUSION

It appears evident that paving of low-volume rural dirt roads is equally beneficial for the road users and the environment. “We have to pay for the road, whether we build it or not” is also valid for low-volume rural roads. This means that if the paving of such a road is economically viable (3, Chapter 2)—subject to proceeding with the required care for the environment—the environmental aspects will also support the decision to pave it.

The inadequate capacity of the main road network has been the primary cause of concern for a long time. However, recently the problems of secondary and low-volume roads were also recognized, and a comprehensive research program is being launched to deal with their operation and management.

REFERENCES

Use of Geotextile-Reinforced Bituminous Seals To Improve Amenity and Environment of Remote Rural Roads

P. G. L. Wolfe

The recent development of geotextile-reinforced bituminous seals on clay pavements for use on remote rural roads to improve amenity and environmental conditions is described. The use of these seals allows substantial economies in the provision of all-weather roads in remote areas and improves access and road safety and reduces materials usage. The design and construction of these types of pavements and a trial using the accelerated loading facility to validate their structural adequacy are described.

Australia is a vast continent with more than 900,000 km of roads, with about 65 percent of these roads being unsurfaced. There are about 600 m of road per person in Australia compared with 300 m in the United States. Funds for provision of all-weather pavements in Australia are at a premium, particularly in the sparsely populated remote rural areas, where traffic volumes are typically less than 100 vehicles per day. In these remote areas, there are large tracts of expansive clay soils, which are particularly difficult to service with cost-effective all-weather pavements.

Most of these areas have average annual rainfalls of significantly less than 600 mm. However, precipitation as low as 3 mm makes these unsealed clay roads untraffickable for days. High summer temperatures, often exceeding 40°C, are common. Figure 1 shows the extent of these expansive clay areas throughout Australia.

Since 1985 the Roads and Traffic Authority New South Wales (RTA) has been developing the use of geotextile-reinforced bituminous seals for the provision of inexpensive all-weather pavements on expansive clay subgrades. These types of pavements are capable of being constructed at between 30 and 40 percent of the cost of conventional pavements with comparable design lives. They also allow considerable improvement in amenity for remote rural communities, substantially reduce the use of local gravels (which are commonly of poor quality), eliminate dust problems, improve road safety, and reduce vehicle wear.

HISTORY

Reinforcing of sprayed bituminous seals is not new. Field Marshal Sir William Slim, reporting in his book Defeat into Victory (Cassell London 1956) on the Burma campaign of World War II, recorded the use of hessian dipped in bitumen and laid on 100 mi of road formation (subgrade only) to support 1,000 army vehicles per day for a year, which included the monsoon season. With the advances in modern geotextiles, their use in this type of application has been tested by RTA since 1985.

THE PROBLEM

The provision of all-weather pavements on expansive clay subgrades is a difficult problem. Clays with linear shrinkage values of 25 percent are not uncommon. Gravel in these areas is usually scarce and of poor quality. Failure modes for traditional unbound granular gravel pavements with a conventional sprayed bituminous chip seal are generally associated with environment rather than load. Failure usually relates to longitudinal cracking resulting from changes in moisture conditions of the expansive clay subgrades, which, in turn, results in surface deformations due to volume changes in the clay. This leads to increased moisture infiltration through the seal, which rapidly accelerates failure during wet periods. Seals under these conditions usually have only limited life, typically 5 to 8 years, because of oxidation of the bitumen from the high summer temperatures and low traffic volumes.

DESIGN

The design of clay pavements with geotextile-reinforced seals comprises a number of considerations.

Pavement Cross Section

The pavement cross section that has been adopted is shown in Figure 2. These types of pavements are generally built on very flat terrain, so the formation is raised above the natural surface using material won from borrow pits outside the road reserve. This is done to eliminate table drains so that water will not lie next to the pavement during the infrequent flooding that occurs in these areas.

Height of Formation

The height that the formation is raised above the natural terrain is a compromise between gaining sufficient height to
minimize moisture infiltration during floods and not having it so high that the formation will desiccate in drought periods. A height at the edge of the shoulder of 600 mm has been adopted in areas subject to intermittent flooding. A height of 300 mm may be suitable in other locations.

**Width of Geotextile Seal**

A seal width of 8.2 m has been adopted. There are two reasons for this. First, 8.2 m can be subdivided into two 3.3-m-wide traffic lanes with 0.8-m shoulders that can be delineated by edgelines. Observations of unsealed clay pavements subject to edge saturation indicate that 0.8 m is close to the limit of infiltration of the wetting front. Second, the geotextiles used are supplied in 4.2-m-wide rolls allowing a 0.2-m overlap in the center of the pavement.

**Pavement Crossfall and Grade**

A pavement crossfall of 4 percent has been adopted because (a) it tends to entice heavy vehicles to travel toward the center (and most moisture stable) area of the pavement and (b) if the clay embankment material swells near the edges as a result of moisture infiltration, adequate crossfall for drainage will still be maintained.

All the pavements built to date have been on very flat grades. Since the geotextile-reinforced seal generally does not bond to the clay pavement but lies on top of it like a heavy blanket, the use of these types of pavements where there is likely to be significant acceleration or deceleration of heavy vehicles must be questioned, because the seal may move.

**Batter Slopes**

Batter slopes of 5:1 have been adopted as a compromise between a flat batter slope to minimize erosion of the formation due to rainfall runoff and the need to minimize earthworks volumes.

**Embankment Compaction**

The embankment material consists of the in situ expansive clay compacted to a minimum level of 95 percent of standard...
maximum dry density. The upper 300 mm of the clay embankment (the pavement) is compacted to a minimum density of 100 percent of standard Proctor maximum dry density (generally in two 150-mm layers). The compaction of the top 300 mm of the clay is critical to the performance of the pavement because it maximizes the bearing capacity of the pavement while reducing its permeability to a very low level. Compaction should be carried out at a moisture content near the long-term equilibrium moisture, generally between 15 and 20 percent for Australian conditions.

Choice of Geotextile

A vast range of geotextiles is now available. For use in sprayed bituminous seals the following characteristics are considered desirable:

- Nonwoven geotextiles are considered better than woven ones because they have more uniform elongation, resist tearing better, and have superior bitumen/fabric adhesion.
- Needle bonded are considered most suitable because heat and resin-bonded joins may become unstable when in contact with hot bitumen.
- Polyamides absorb water, lose strength, and tend to become brittle; polyvinyls may be subject to bacterial attack; and polylefines (polypropylene and polyethylene) burn easily (melting point typically 165°C), exhibit high creep, and are subject to ultraviolet light deterioration. Polymers, which do not burn easily (melting point typically 250°C), absorb only small amounts of water, and are less sensitive to ultraviolet light, are the most suitable. Polypropylene can be used if bitumen emulsion is used in the sealing process.

Nonwoven needlepunched polyester is therefore the preferred material. Recommended properties are given in Table 1.

Geotextile-Reinforced Seal

The application of the geotextile reinforced seal involves the following steps:

- A bitumen prime coat is applied to the clay surface, which has been swept and lightly watered. The binder is a Class 170 hot bitumen incorporating up to 3 percent cutter and is applied at a (cold) application rate of about 0.8 to 1.0 L/m², depending on the surface absorption characteristics of the pavement.
- The geotextile is then rolled out onto the primed surface using a mechanical applicator, which follows close behind the sprayer.
- A scatter coat of 7 mm aggregate is spread on top of the geotextile and rolled to bring the bitumen to the surface of the geotextile and to provide a tack-free surface for construction traffic.
- A hot bitumen seal (Class 170 bitumen), designed in accordance with RTA Seal Design Method (J), is then applied. [The usual (cold) application rate is about 1.3 L/m². If a polymer-modified binder is used, the usual application rate should be about 1.9 L/m².] The use of polymer-modified binders will obviously increase the initial cost but has the advantages of better aggregate retention and a reduced rate of binder oxidation. Cutter may be added to the binder in accordance with normal seal design procedures (J).
- Ten mm of cover aggregate is then spread and immediately rolled to cover the area sprayed.
- The behavior of the seal is dependent on the absorption of the binder into the pavement and the geotextile, which is often difficult to estimate. Allowances of between +0.5 and +0.8 L/m² are not uncommon. If the seal is deficient in binder, it may require an enrichment.

This provides a very substantial seal with a heavy application of binder, which is quite robust and resistant to oxidation resulting from high summer temperatures and low traffic volumes.

Traffic Markings

Edgelines only are marked on the pavement to encourage travel away from the edges of the pavement, and guideposts are placed 800 mm in from the edge of the seal to stop wheels traveling on the outer edges of the sealed area.
Pavement Maintenance

There is a need for a regular enrichment program to counter the effects of oxidation of the bitumen binder. If polymer-modified binders are used, this should decrease the frequency of the required enrichment.

Patching of the geotextile seal may be necessary and can be achieved by squaring any hole in the geotextile, cutting corner slits, placing a geotextile patch that "underlaps" the surrounding geotextile by about 150 mm, and then spraying the patch with bitumen emulsion and applying a 7-mm aggregate. Commercially available self-adhesive rubberized bitumen membranes may also be used.

PAVEMENT PERFORMANCE

The long-term performance of clay pavements with geotextile seals depends on

- The structural capacity of the pavement to withstand the traffic loading,
- The environmental degradation of the seal and the pavement as a result of the temperature and moisture regime in which the pavement must perform, and
- The amount of accelerating and braking of heavy vehicles.

Because of the very fine nature of the clay, the geotextile-reinforced seal does not bond to the clay but sits on top of it. If there is heavy braking or acceleration, there may be local movements of the seal. If these areas can be identified during design, the use of a thin layer of gravel over the clay will ensure bonding of the seal.

The first trials of clay pavements with geotextile-reinforced seals were carried out on the Cobb Highway in western New South Wales in 1985. These trial sections are still in use and have suffered imperceptible structural damage to date and have only received routine maintenance treatment. However, there has been insufficient traffic to determine the structural adequacy over a design life of 20 years.

BREWARRINA ACCELERATED LOADING FACILITY TRIAL

With the popularity of clay pavements with geotextile seals increasing, it was decided to conduct an accelerated loading test using the accelerated loading facility (ALF). The ALF is a relocatable road-testing machine that applies full-scale rolling wheel loads to a test pavement. The ALF was designed and manufactured by RTA and is owned and operated by the Australian Road Research Board in cooperation with AUSTROADS, the national road authority coordinating group. ALF has been used continuously since 1984 in a series of nationally coordinated pavement trials (2).

The trial was conducted on a remote arterial road in the northwest of New South Wales near the town of Brewarrina. The aims of the Brewarrina ALF trial were (a) to determine the structural adequacy of pavements consisting of geotextile-reinforced seals over prepared clay subgrades and (b) to gain knowledge of how distance of the traffic loading from the edge of the geotextile seal and the presence of water adjacent to the edge of the pavement affect the performance of these pavements.

The trial was carried out between July and December 1991, and detailed reporting is presented by Walter (3), Sharp and Johnson-Clarke (4), and Sharp and Walter (5).

For most expansive clay pavements in remote areas of New South Wales, the design traffic for a 20-year design life would be less than 200,000 equivalent standard axles (ESAs). A standard axle in Australia is 8.2 tonnes on a single axle with dual wheels. To test the worst possible case, with water adjacent to the edge of the seal, dams were built adjacent to some of the test pavements as shown in Figure 3.

The results of the tests on the clay pavement sections are summarized in Table 2.

FIGURE 3 Brewarrina ALF test pavement layout.
TABLE 2 Summary of Brewarrina ALF Trial Results

<table>
<thead>
<tr>
<th>Position of Loaded Wheel</th>
<th>Rut depth</th>
<th>ESAs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Centre of dry pavement</td>
<td>7mm</td>
<td>20,000</td>
</tr>
<tr>
<td>1.9m from pavement edge</td>
<td>9.4mm</td>
<td>13,800</td>
</tr>
<tr>
<td>with water at edge of seal</td>
<td>35mm</td>
<td>220,000*</td>
</tr>
</tbody>
</table>

* Indicates end of testing.

A further trial was carried out with the ALF at an angle to the longitudinal direction of the pavement and with water at the edge of the pavement, to try and get the loaded wheel closer to the edge of the pavement. This resulted in substantial deformations on the outer 1 m of the pavement after only 150 load cycles. This illustrates the sensitivity of the clay pavement to moisture increases and indicates that these types of pavements would not be applicable in areas subject to inundation during flood times. However, from Table 2, the substantial structural capacity of the clay pavements was demonstrated by the ALF.

The transverse distribution applied by the ALF was a normal distribution of width 1.4 m. This is applicable to normal lane widths of about 3.5 m on reasonably heavily trafficked roads. Recent preliminary measurements on remote rural roads where there is little oncoming traffic and very good sight distances (6) indicate that transverse distribution of wheel loads on low-trafficked roads is substantially wider than 1.4 m. This means that the design ESAs used for this type of road may be decreased significantly.

SAFETY CONSIDERATIONS

Where there are long lengths of unsealed pavements that create safety problems as a result of dust hazards making overtaking maneuvers dangerous during dry periods, geotextile-reinforced seals have been used in short lengths to provide safe overtaking opportunities. In these situations, appropriate signs are erected to enable motorists to maximize the safety of overtaking maneuvers. A typical signposting scheme is shown in Figure 4.

This type of scheme has been implemented in remote areas of western New South Wales and is a cost-effective means of improving road safety on remote roads, particularly where tourist and commercial (e.g., stock haulage) traffic are mixed.

CONCLUSION

The cost of providing an all-weather pavement using clay pavements and geotextile-reinforced seals is approximately 30 to 40 percent of a conventional pavement with 300 mm of gravel and a normal bituminous seal.

REFERENCES

PART 2

Modern Timber Bridges
Development and Evaluation of the Teal River Stress-Laminated Glulam Bridge

MICHAEL A. RITTER, JAMES P. WACKER, KIM STANFILL-MCMILLAN, AND JAMES A. KAINZ

The Teal River bridge was constructed in late 1989 in Sawyer County, Wisconsin, as part of the demonstration timber bridge program of the U.S. Department of Agriculture Forest Service. The bridge is a stress-laminated deck structure with a 9.91-m length and a 7.23-m width. The design is unique in that it is the first known stress-laminated timber bridge in the United States to be constructed of full-span glued-laminated timber beams rather than the traditionally used sawn lumber laminations. The performance of the bridge was continuously monitored for 2 years, beginning at the time of installation. The performance monitoring involved gathering data relative to the moisture content of the wood deck, the force level of stressing bars, the deck dead load deflection, and the behavior of the bridge under static-load conditions. In addition, comprehensive visual inspections were conducted to assess the overall condition of the structure. On the basis of 2 years of field evaluations, the bridge is performing well with no structural or serviceability deficiencies.

In 1988 the U.S. Congress passed the Timber Bridge Initiative legislation. The objective was to establish and annually fund a national timber bridge program to provide effective utilization of wood as a structural material for highway bridges. Responsibility for the development, implementation, and administration of the timber bridge program was assigned to the USDA Forest Service. A key element of this program is a demonstration bridge program, which provides matching funds to local governments to demonstrate timber bridge technology through the construction of demonstration bridges (1).

As a national wood utilization research laboratory within the USDA Forest Service, the Forest Products Laboratory (FPL) has taken a lead role in assisting local governments in evaluating the field performance of demonstration bridges, many of which use design innovations. This has involved the development and implementation of a comprehensive national bridge monitoring program, which collects, analyzes, and distributes information on the field performance of timber bridges. This information provides a basis for validating or revising design criteria to improve efficiency and economy in bridge design, fabrication, and construction.

This paper describes the development, design, construction, and field performance of the Teal River bridge located in Sawyer County in northwestern Wisconsin. The bridge, built in 1989, is a two-lane, single-span, stress-laminated deck with a length of 9.91 m. The bridge design is the first known U.S. application that uses full-span structural glued-laminated (glulam) timber beams in a stress-laminated deck. In 1991 this bridge design was awarded first place in a National Timber Bridge Design Competition in the "Under 12-m Individual Span Vehicular Bridge" category.

BACKGROUND

The Teal River bridge site is located approximately 32 km east of Hayward, in Sawyer County, Wisconsin. It is on County Highway S, a two-lane paved road that crosses the Teal River. This road is located within the boundary of the Chequamegon National Forest and provides access to several popular recreation areas. In addition, the road is on the Chequamegon National Forest transportation network and is a primary route for logging traffic. The estimated average traffic over this section of the road is 100 vehicles per day.

The Teal River bridge was originally constructed in 1925 and consisted of steel stringers with a concrete deck supported by concrete abutments. The bridge was 10.37 m long and 4.88 m wide and included a rail system constructed of steel angles. In 1988, inspections of the bridge indicated that the concrete deck was in poor condition and the steel girders were badly corroded, although not to a point where restricted load limits were required. In addition, the railing system was substandard, and the narrow bridge width on a two-lane road raised safety concerns. It was apparent to the Sawyer County Highway Department that major rehabilitation or replacement of the structure would be required in the near future.

Subsequent to the bridge inspection, Sawyer County officials determined that the Teal River bridge would be replaced. Through a cooperative effort involving Sawyer County, the North Twenty Resource Conservation and Development Council, and the Chequamegon National Forest, a project proposal was submitted to the USDA Forest Service for partial funding of the Teal River bridge replacement as a demonstration bridge under the Timber Bridge Initiative. The proposal included a stress-laminated deck constructed of red oak sawn lumber harvested from Wisconsin forests. In 1989 the project was approved as proposed, and matching funds were provided through the USDA Forest Service Timber Bridge Information Resource Center in Morgantown, West Virginia. In finalizing project plans, it was found that neither red oak nor red pine lumber was readily available in the required size. Consequently, FPL was contacted for assistance in developing material options for the bridge, where the use of wood products native to Wisconsin was a primary consideration. Preliminary investigations indicated that glulam timber beams presented the best alternative for design. Hence, red pine lumber...
could be laminated with structural waterproof adhesives to form beams of the size required for the bridge laminations.

DESIGN, CONSTRUCTION, AND COST

The design and construction of the Teal River bridge was completed by the Chequamegon National Forest engineering staff in cooperation with the Sawyer County Highway Department officials. Assistance was provided by the bridge design office of the Forest Service Eastern Regional Office and FPL. An overview of the design, construction, and cost of the bridge superstructure is presented.

Design

Design of the Teal River bridge was a two-part process involving the development of the glulam timber beams followed by the design of the bridge superstructure. As with most stress-laminated timber bridge decks, it was anticipated that stiffness rather than strength would be the primary design concern. To serve as a basis for designing the glulam timber beams, a preliminary analysis of the Teal River bridge indicated that an acceptable deck depth could be obtained if a minimum design modulus of elasticity (MOE) of 11 204 MPa could be achieved. Analysis conducted at FPL indicated that a glulam timber beam manufactured entirely of red pine was feasible. However, little information was available on the properties of red pine lumber, and no information was available on red pine glulam timber. Therefore, the FPL engineers proposed that southern pine lumber be combined with red pine lumber to provide additional beam stiffness. No design information on southern pine-red pine glulam timber beams was available in current American Institute of Timber Construction (AITC) standards (2). However, the concept of developing beams using lower stiffness species for the inner lumber laminations and higher stiffness species for the outer lumber laminations had been used for other species in this standard. The same concept was used to develop a proposed beam design with southern pine outer laminations and red pine inner laminations. The proposed beam design was subsequently approved by AITC and was used in the Teal River bridge (Figure 1).

After the glulam timber design values were developed, design of the Teal River bridge was completed by the engineering staff of the Chequamegon National Forest using criteria developed at the University of Wisconsin—Madison and FPL (3). The design geometry of the deck provided for a 9.91-m length, a 7.32-m width, and a 350-mm thickness (Figure 2). This required the use of 91 southern pine-red pine glulam timber beams, each measuring approximately 79 mm wide. The outside beam along each deck edge was designed to be red oak glulam timber. The red oak would provide additional strength in distributing the force in the stressing bars into the deck without damaging the southern pine–red pine beams. All glulam beams were used as bridge laminations to form a continuous deck and hereafter are referred to as beam laminations.

All beam laminations were designed to be continuous between supports, without butt joints. The deck was also provided with a curb and bridge rail system to meet AASHTO static-load design requirements (4). Following fabrication, all wood components were specified to be pressure treated with pentachlorophenol in accordance with American Wood Preservers’ Association Standard C14 (5).

The stressing system for the Teal River bridge was designed to provide a uniform compressive stress of 0.69 MPa between the beam laminations. It was assumed that approximately 60 percent of this compression will be lost during the lifetime of the bridge as a result of transverse stress relaxation of the wood laminations. The remaining 40 percent, or 0.28 MPa of compression between the laminations, provides a safety factor greater than 2.0 against relative laminate movement caused by transverse shear or bending. To provide this interlaminar compression, 25-mm-diameter high-strength stressing bars were spaced 1.12 m on center. The bars were specified to comply with the requirements of ASTM A722 and to provide a minimum ultimate tensile strength of 1034 MPa. The bar anchorage system was the discrete plate anchorage system, consisting of 305- by 305-mm steel-bearing plates with 102- by 165-mm steel anchorage plates. To provide protection from deterioration, all steel components were galvanized, including hardware, stressing bars, and anchorage plates.

Construction

Construction of the Teal River bridge was completed by Sawyer County and Chequamegon National Forest personnel in fall 1989. Following work on the approach roadway and widening of the existing concrete abutments, construction of the bridge superstructure began November 14 and was completed November 15. However, several additional days were required for construction of the bridge railing and approach railing. During this period, the construction site was subjected to inclement weather conditions, including snow and cold temperatures. Although the weather tended to slow the construction, it was completed on schedule with little difficulty.

Construction of the bridge began with the arrival of the beam laminations at the bridge site. The 91 southern pine-red pine beam laminations were transported to the site on a flatbed trailer in banded panels consisting of 15 to 16 beam laminations per panel. Each beam lamination measured 79

![Diagram](https://via.placeholder.com/150?text=Figure+1+Southern+pine-red+pine+beam+laminations+used+for+Teal+River+bridge.)
mm wide by 350 mm deep by 9.91 m long. Panels were placed on the abutments by a crane, bands were removed, and the beam laminations were positioned by the work crew. After placement of the southern pine—red pine beam laminations, a red oak beam lamination was placed along each deck edge. During placement, it was discovered that the abutment configuration restricted the bridge width and would not allow placement of all the beam laminations. Consequently, one southern pine—red pine beam lamination was removed, resulting in a finished bridge width of approximately 7.23 m (design width was 7.32 m).

After all beam laminations were in place, steel stressing bars were manually inserted through the predrilled holes in the beam laminations; bearing plates and anchor plates were installed, and nuts were hand tightened. Several stressing bars were then partially tensioned to bring all beam laminations in contact, and the initial deck stressing began. This was accomplished with a hydraulic jacking system consisting of a hydraulic pump, a single hollow core jack, and a stressing chair (3). Using this system, force applied by the jack is transferred through the stressing chair to the bar bearing plate. The bar is pulled away from the deck until the design force (269 kN) is reached. Then, the anchorage nut is tightened with a wrench to lock off the tension force in the bar. Starting at one bridge end, each stressing bar of the Teal River bridge was tensioned in this manner. After all bars were tensioned, each was retensioned to ensure that the stress level in the deck was uniform and at the required design level.

Following the initial stressing, the timber curb and rail system was installed, and a glulam timber approach rail was constructed. Approximately 1 week after the initial stressing, the bridge was restressed to compensate for anticipated losses in the bar force. Approximately 7 weeks after the second deck stressing, the third and final deck stressings were completed. An asphalt wearing surface was subsequently applied in June 1990. The completed bridge is shown in Figure 3.
Costs for the design, fabrication, and construction of the Teal River bridge superstructure and rail system were $4,000 for survey and design, $26,485 for materials, and $5,400 for labor and equipment, for a total cost of $35,885. On the basis of a total deck area of 72.5 m², the cost per square meter was approximately $495.

EVALUATION METHODOLOGY

To evaluate the structural performance of the Teal River bridge, Sawyer County representatives contacted FPL for assistance. By agreement, a bridge monitoring plan was developed by FPL and implemented as a cooperative research effort with Sawyer County and the Chequamegon National Forest. The plan called for stiffness testing of the beam laminations before bridge construction and for performance monitoring of the deck moisture content, bar force in stressing bars, bridge creep, and load test behavior and for condition assessments of the structure for the first 2 years in service. The evaluation methodology used procedures and equipment previously developed (6) and is discussed in the following sections.

Lamination Stiffness

The glulam timber beam developed for the beam laminations of the Teal River bridge was a new combination. Thus, stiffness tests were completed to verify design assumptions. At the manufacturing plant, MOE tests were performed on the lumber before gluing and on the completed glulam timber beams before preservative treatment. Three methods were used to determine MOE values (7). The first, known as the static-load method, involved placing a known load at the lumber or beam midspan and measuring the deflection with a dial gauge. The second, known as the transverse vibration technique, involved striking the lumber or beam to induce a transverse vibration and measuring the natural frequency with a computer (8). The third used stress-wave technology and involved inducing a longitudinal stress wave in the lumber or beam and measuring the time of flight of the wave along the lamination length.

Moisture Content

Changes in the moisture content level of stress-laminated timber decks can significantly affect the performance of the structure. If moisture decreases, the deck can shrink, resulting in a decrease in stressing bar force. If moisture increases, swelling of the timber can occur and cause an increase in stressing bar force. Changes in moisture content level can also affect the deck stiffness, creep, and transverse stress relaxation.

To measure the moisture content of the Teal River bridge deck, an electrical resistance moisture meter with 76-mm pins was used. Measurements were obtained on a monthly basis by driving probe pins into the deck underside at depths of 51 to 76 mm, recording the moisture content value from the unit, then adjusting the values for temperature and wood species. At the 76-mm maximum pin penetration depth, measurements were taken in the lower southern pine portion of the beam laminations.

Bar Force

For stress-laminated bridges to perform properly, an adequate level of interlaminar compression must be maintained between the bridge laminations. This compression is placed in the bridge by tensioning the stressing bars to high levels and maintaining a portion of this force during the life of the structure. Thus, the force level in the bars provides a direct indication of the interlaminar compression in the bridge. For the Teal River bridge, the initial interlaminar compression of 0.69 MPa required a force in each stressing bar of 269 kN. If the force level decreases more than approximately 80 percent (i.e., less than 20 percent remaining), structural and serviceability problems can occur.

To monitor bar force, load cells developed by FPL were installed on two of the nine stressing bars when the bridge was assembled. The cells consisted of a steel cylinder that was placed between the stressing bar bearing plate and anchorage plate (6). Each cell was provided with two 90-degree strain gauge rosettes that measured the strain in the load cell. Strain
measurements were then converted to force levels to determine the force remaining in the bar. Load cell measurements were obtained by connecting a portable strain indicator to a plug on the load cell body. Measurements were taken on a biweekly basis for the first year and monthly thereafter. Approximately midway through the monitoring period, 1 year after bridge construction, the load cells were unloaded and checked for zero balance shift.

**Creep**

As a structural material, wood can deform permanently as a result of long-term sustained loads. For stress-laminated bridges, creep caused by structure dead load is an important consideration, because excessive creep can result in a sag in the superstructure (8). Creep of the Teal River bridge was measured on a monthly basis with a displacement rule attached to the deck underside at midspan. Vertical movement over time was recorded relative to a stringline attached near the abutments. In addition, a surveying level and rod were used periodically to confirm stringline data.

**Load Test Behavior**

Static-load testing of stress-laminated bridges is an important part of a comprehensive bridge monitoring program. The information obtained from these tests is used to refine and improve design procedures and evaluate the effects of various design variables on bridge performance. To determine the load test behavior of the Teal River bridge, load tests were conducted 7 and 17 months after installation. Each test consisted of positioning fully loaded trucks on the bridge deck and measuring the resulting deflections at a series of locations along the bridge centerspan, quarter points, and abutments. Measurements of bridge deflections were taken before testing (unloaded), for each load position, and at the conclusion of testing (unloaded).

**Load Test 1**

For the first load test on June 19, 1990, the vehicle used was a three-axle loaded dump truck with a gross vehicle weight of 355.2 kN. The vehicle was positioned longitudinally on the bridge so that the centroid of the vehicle aligned with the bridge centerspan for each of three transverse load positions (Figure 4). Measurements of bridge deflections from an 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.

**Load Test 2**

The second load test, on April 26, 1991, involved two test vehicles: Truck 11 with a gross vehicle weight of 265.1 kN and Truck 13 with a gross vehicle weight of 266.4 kN. For this test, the vehicles were positioned longitudinally with the two rear axles centered over the bridge centerspan (front axles were off the bridge), and three transverse load positions were used (Figure 4). Measurements of bridge deflections were obtained by suspending calibrated rules from the deck underside and reading values to the nearest 1.5 mm with a surveyor’s level.

**Predicted Behavior Under HS 20-44 Loading**

Previous research showed that stress-laminated decks can be accurately modeled as orthotropic plates (9). To further analyze the behavior of the Teal River bridge, an orthotropic model currently being developed and verified at FPL was used to predict the deflection of AASHTO HS 20-44 loading and evaluate changes in bridge stiffness.

**Condition Assessment**

The general condition of the bridge was assessed initially at the time of installation, twice during the load testing, and finally during a site visit near the end of the monitoring period. These assessments involved visual inspections, measurements, and photographic documentation of the bridge condition, specifically the condition of the timber deck and rail system, asphalt wearing surface, stressing bars, and anchorage systems.

**RESULTS AND DISCUSSION**

The performance monitoring of the Teal River bridge extended from November 1989 through October 1991.

**Lamination Stiffness**

Results of MOE tests using the transverse vibration technique on southern pine lumber before gluing indicated an average MOE of 14 262 MPa for No. 1 dense material and 13 298 MPa for No. 2 dense material. In both cases, average values were slightly greater than assumed design values. For red pine, lumber MOE tests using the same method provided an average MOE of 10 266 MPa, which was approximately 35 percent greater than assumed design values.

Tests of 30 laminated beams using the transverse vibration technique resulted in an average MOE of 12 265 MPa. Additional testing at the plant using the static-load method resulted in an average MOE of 12 195 MPa for 24 laminated beams. MOE tests using the stress-wave technique did not provide reliable data and were not used for evaluation.

The average MOE of the laminated beams exceeded the target value of 11 204 MPa by approximately 10 percent. This was because lumber properties for both southern pine and red pine exceeded those assumed in the original design.

**Moisture Content**

The beam laminations were initially installed at an average moisture content of less than 10 percent. Since installation,
moisture content gradually increased to an average level of approximately 13 percent at the conclusion of the monitoring period. Moisture content fluctuated following seasonal climate changes, with a maximum average moisture content of approximately 15 percent occurring in fall 1990. These changes will continue over the life of the structure and were generally most apparent in the outer 25 to 76 mm of the deck, where moisture measurements were taken. Moisture changes in the interior portion of the deck, where the moisture content will continue to increase until an equilibrium level is reached, were gradual. It is anticipated that the average moisture content of the deck will eventually stabilize at an equilibrium value of 18 to 20 percent (10), although short-term seasonal changes will continue, primarily in the outer 51 to 76 mm of the deck.

**Bar Force**

Bar tension force was measured with load cells on two stressing bars and averaged. The first two stressings were at the design force of 269 kN. The final stressing, which occurred about 8 weeks after installation, was approximately 10 percent greater than the design force level. As is typical of stressed-laminated decks, the rate of bar force loss as a result of trans-
verse stress relaxation decreased substantially with each restressing.

The observed minor fluctuations in bar force were a result of moisture changes and stress relaxation in the deck. However, force losses were minimal, and the average bar force was within 10 percent of the design force at the end of the monitoring period. This can be attributed to several factors, the most significant of which was the initial low moisture content level of the beam laminations. As the deck slowly gained moisture in reaching an equilibrium moisture content, the wood swelled slightly, which tended to offset force losses as a result of transverse stress relaxation. Other stress-laminated decks installed at relatively high moisture content levels had bar force losses of as much as 80 percent during 2 years (11). The Teal River bridge vividly illustrates the advantage of low wood moisture content levels at the time of construction in reducing the rate of bar force loss.

Creep

The beam laminations for the Teal River bridge were manufactured with a positive camber of approximately 51 mm. After the dead load deflection of about 10 mm, measurements indicated that approximately 3 mm of vertical creep occurred at centerspan during 2 years. At the conclusion of the monitoring period, approximately 38 mm of positive camber remained in the deck at centerspan.

Load Test Behavior

Results for both load tests and the predicted response of the bridge under AASHTO HS20-44 loading are presented. In each case, transverse deflection measurements are given at the bridge centerspan as viewed from the south end (looking north). To aid visual interpretation, deflection values are presented as fourth-order polynomial curve fits to the measured data points. For each load test, no permanent residual deformation was measured at the conclusion of the testing. In addition, movement at either of the abutments was not detected.

Load Test 1

Transverse deflection values for Load Test 1 are shown in Figure 5. For Load Position 1, the maximum deflection of 21 mm occurred under the outside wheel line, 915 mm from the downstream deck edge. For Load Position 2, the maximum deflection of 22 mm was measured 254 mm from the outside wheel line, toward the upstream deck edge. For Load Position 3, the maximum deflection of 22 mm occurred under the outside wheel line, 915 mm from the upstream deck edge.

Load Test 2

Transverse deflection values for Load Test 2 are shown in Figure 5. With a single truck on the bridge, both Load Positions 1 and 2 produced a maximum deflection of 15 mm approximately midway between the truck wheel lines. With both vehicles on the bridge, the maximum deflection for Load Position 3 was 24 mm under the inside wheel line of Truck 11.

Predicted Behavior Under HS20-44 Loading

On the basis of an analysis of both load tests using an FPL computer model, the maximum deflection of the Teal River bridge subjected to two lanes of HS20-44 loading was estimated at 25 mm. Further analysis of Load Test 2 compared with Load Test 1 indicated that for equivalent HS20-44 loading, no significant decrease occurred in bridge stiffness during the monitoring period. This was expected because neither a significant decrease in bar force nor an increase in deck moisture content occurred during the monitoring period.

Condition Assessment

Condition assessments of the Teal River bridge indicated that structural and serviceability performance were good. Inspection results for specific items follow.

Wood Components

Wood components of the bridge showed no signs of deterioration, although minor checking was evident on rail members exposed to wet-dry cycles. Checking was most pronounced in the end grain of the southern pine glulam timber rail posts. This would likely have been prevented if a bituminous end-grain sealer or a metal cap had been placed at the time of construction. In addition, the top of the bridge rail showed minor checking, but the checks did not appear to penetrate the preservative treatment envelope of the member. Inspection showed no evidence of wood preservative loss and no preservative or solvent accumulations on the wood surface. The red oak edge beams showed a lightening in color from a medium tan to a very light tan as a result of exposure to sunlight.

Wearing Surface

Inspection of the asphalt wearing surface indicated minor transverse cracking in a random pattern. This was attributed to a deficiency in the asphalt mix or application procedures, because the same cracking was observed on both approach roadways, which were paved at the same time as the bridge deck. Aside from the minor cracking, the asphalt was in good condition and showed no other signs of distress.

Anchorage System

The stressing bar anchorage system performed as designed with no significant signs of distress. Inspection indicated no crushing of the discrete plate anchorage into the outside red oak beams and no measurable distortion in the bearing plate. A very gradual compression deformation was noted along the
Load Test 1

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Load Position 3

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</table>

FIGURE 5 Load test transverse deflections, measured at the bridge centerspan. Bridge cross sections and vehicle positions are shown to aid interpretation and are not to scale.

deck edge for a distance of several feet on either side of several anchorages. This deformation was difficult to detect visually and was likely the result of transverse stress relaxation in the beam laminations.

Stressing Bars and Hardware

The exposed steel stressing bars and hardware showed no visible signs of corrosion except at the ends of stressing bars. At bar ends, minor corrosion appeared where the galvanized coating had been stripped from the bar, exposing uncoated steel. This occurred because the nuts were not adequately oversized to compensate for galvanizing and were forced on the bars during construction. This problem would not have occurred if nuts had been properly oversized to compensate for galvanizing or a cold galvanizing compound had been applied to the bar to replace the removed coating.

CONCLUSIONS

After 2 years in service, the Teal River bridge is exhibiting excellent performance and should provide many more years of acceptable service. On the basis of the extensive monitoring conducted since bridge fabrication, the following conclusions are given:

1. It is both feasible and economically practical to manufacture structural glulam timber beams for bridge applications using a combination of red pine and southern pine lumber. The evaluation of this project indicates that beams could prob-
ably be manufactured entirely from red pine, provided that appropriate quality control measures are implemented to ensure stiffness requirements for the specific design.

2. Stress-laminated decks can be constructed using glulam timber beams for the deck laminations. The ability to manufacture the glulam timber beam laminations as continuous members greatly facilitates transportation and construction because butt joints are not required.

3. The use of red oak outside edge laminations facilitates good performance of discrete plate stressing bar anchorages. Red oak provides sufficient strength to adequately distribute the bar force into the deck without wood crushing or anchor plate deformation.

4. The average trend in deck moisture content indicates that global moisture content changes are occurring very slowly, with an average increase of approximately 3 percent during the 2 years. Cyclic seasonal variations in moisture content are occurring more rapidly and at greater magnitudes in the outer 51 to 76 mm of the exposed deck.

5. Stressing bar force remained at a relatively high level during the 2-year monitoring, with less than a 10 percent average decrease in bar force below the initial level. This is attributable primarily to low average moisture content levels of the bridge laminations at installation, which were less than the anticipated equilibrium moisture content for the site. Loss in bar force caused by transverse stress relaxation in the wood is minimized by dimensional increases in the deck as moisture content increases toward an equilibrium level. The lower moisture content level also reduces the rate of stress relaxation within the deck and in the vicinity of bar anchorage plates.

6. Creep of the bridge deck is minimal with approximately 3 mm of vertical displacement during the 2 years. A positive camber of approximately 38 mm remains at centerspan.

7. Load testing and analysis indicate that the Teal River bridge is performing as a linear elastic orthotropic plate when subjected to highway loading. The maximum deflection caused by two lanes of AASHTO HS20-44 loading is estimated to be 25 mm.

8. The deck stiffness is not appreciably changed. This is attributable to the high level of prestress maintained in the deck during the monitoring period.

9. Wood checking is evident in the exposed end grain of bridge rail posts and other components. It is likely that this would not have occurred if a bituminous sealer or metal cap had been applied to the end grain at the time of construction.

10. The ends of some stressing bars show signs of minor corrosion at locations where the galvanizing was removed during construction. This would not have occurred if the stressing nuts had been oversized to compensate for the thickness of the galvanized coating.

REFERENCES

Behavior of a Bilayer Reinforced Stressed Timber Bridge Deck Under Static and Dynamic Loads

William G. Buttlar and Ralph R. Mozingo

Stressed timber bridge decks are constructed by pressure laminating thin wooden members into solid slab, using a simple posttensioning system. Economic studies indicate that finding ways to lower timber costs is important in making this type of structure more cost competitive. Recently, stressed timber decks reinforced with steel sandwich plates have been introduced. They are inherently more stiff, ductile, and resistant to creep than their non-reinforced predecessors. A "bilayer" reinforced stressed timber deck configuration, using two separate layers of relatively small, low-cost deck boards, was investigated. Full-scale laboratory testing was performed to measure stiffness, efficiency, and material stresses of a bilayer prototype deck constructed with combinations of three prestressing pressures and six steel reinforcement plate levels. Both static and dynamic loads were applied to the deck. The bilayer prototype demonstrated predictable, orthotropic behavior for decks containing as low as 1.64 percent steel by volume. In a nondimensional comparison, the bilayer was found to be slightly less efficient than single-layered decks, but comparable. Application of heavy static and dynamic loads did not introduce any measurable interlaminate slip. Whereas an economical bilayer configuration is possible when steel sandwich plates are present, additional testing is needed to further establish comprehensive design criteria for this type of structure.

The practice of friction laminating wood originated in Canada when deteriorating nail-laminated bridge decks were rehabilitated with transverse posttensioning (1). Prestress pressure was applied perpendicular to laminations by tensioning high-strength steel rods placed transversely on the top and bottom of the deck and anchored on deck sides. This posttensioning system caused high interlaminate friction between adjacent timbers and increased load distribution capabilities of the deck. The stressed timber deck was eventually introduced into new bridge construction and was refined to have a single row of butt joints. Stiffness of the model deck with shorter deck timbers was investigated by increasing the number of "butt joints," which weakened the overall stiffness contribution of wood in the deck. A butt joint is a longitudinal gap in timbers present on all stressed decks more than 6.1 m (20 ft) in length, and the arrangement of butt joints in a stressed timber deck is commonly referred to as the butt pattern. Stiffness of the model deck with shorter timbers (more butt joints) was found to converge to stiffness of the original deck as more steel strips were added.

Full-scale testing of a steel-reinforced stressed deck at The Pennsylvania State University was funded by a grant from The Ben Franklin Technology Center. A 12.2-m (40-ft) half-lane prototype was constructed and tested (2). The prototype, called Butt Pattern A (Figure 1a, Table 1), consisted of unseasoned, visually graded No. 2 mixed red and white oak timbers along with high-strength, corrosion-resistant (ASTM A588) steel sandwich plates 9.5 mm x 356 mm x 12.2 m (% in. x 14 in. x 40 ft 0 in.). The sandwich plates were not galvanized. It was found that when the prototype deck contained about 5 percent steel by volume, stiffness was doubled, bearing plates did not significantly deform facia timbers of the hardwood deck, and lower-grade lumber was used without introducing structural deficiencies.

The scarcity of wide, long timbers in most states prompted investigation of the use of shorter (and, thus, more economical) deck timbers. Load testing of a modified single-layer deck, or Butt Pattern B, was performed (Yannuzzi, unpublished data, 1990). Construction of the new deck merely required doubling butt joints in the existing deck (Figure 1b). To consider worst-case placement of butt joints, additional joints were added along the same transverse lines as previous joints. Therefore, along any given row of butt joints, 50 percent of timber was absent, double the amount in Butt Pattern A.

The Pennsylvania State University, University Park, Pa. 16802.
TABLE 1 Comparison of Various Stressed Timber Bridge Deck Configurations

<table>
<thead>
<tr>
<th>Layer Configuration</th>
<th>Trout Rd. Bridge(^a)</th>
<th>Butt Pattern A</th>
<th>Butt Pattern B</th>
<th>Butt Pattern C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood Species</td>
<td>Douglas Fir</td>
<td>Mixed red/white oak</td>
<td>Mixed red/white oak</td>
<td>Mixed red/white oak</td>
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<tr>
<td>Grade of Wood</td>
<td>No. 2</td>
<td>No. 2</td>
<td>No. 2</td>
<td>No. 2</td>
</tr>
<tr>
<td>Cross-sectional dimensions</td>
<td>102 mm x 406 mm (4” x 16”)</td>
<td>51 mm x 356 mm (2” x 14”)</td>
<td>51 mm x 356 mm (2” x 14”)</td>
<td>51 mm x 165 mm (2” x 6.5”)</td>
</tr>
<tr>
<td>Range of timber lengths</td>
<td>up to 5.5 m (up to 18”)</td>
<td>2.1 m to 5.3 m (6’-11” to 17’-5”)</td>
<td>2.1 m to 3.2 m (6’-11” to 10’-5”)</td>
<td>1.8 m to 3.5 m (6’-0” to 11’-4”)</td>
</tr>
<tr>
<td>Approx. wood MOE(^b)</td>
<td>9,715 mPa (1.41 x 10(^6) psi)</td>
<td>10,300 mPa (1.49 x 10(^6) psi)</td>
<td>10,300 mPa (1.49 x 10(^6) psi)</td>
<td>11,100 mPa (1.61 x 10(^6) psi)</td>
</tr>
<tr>
<td>Approx. wood MOR(^c)</td>
<td>51.0 mPa (7,400 psi)</td>
<td>72.6 mPa (10,540 psi)</td>
<td>72.6 mPa (10,540 psi)</td>
<td>69.6 mPa (10,100 psi)</td>
</tr>
</tbody>
</table>

\(^a\) Modulus of Elasticity \\
\(^b\) Modulus of Rupture \\
\(^c\) Values of MOE and MOR for Trout Road Bridge were estimated from the Wood Handbook (9). Properties for butt patterns A, B, and C were obtained from static bending tests in conformance with ASTM D143.

Without steel, Butt Pattern B stiffness was found to be 77 percent of the original stiffness of Butt Pattern A. With 15 steel plates in the deck, Butt Pattern B exhibited 95 percent of the stiffness of Butt Pattern A. Thus, the problem of shorter timber lengths was solved. But what about shorter timber widths? Could timber from small- and medium-diameter trees be used to form layers of stressed decks?

A new concept in stressed timber bridge design involves the use of a “bilayer” wooden deck with steel sandwich plates. A model study (Mozingo, unpublished data, 1988) conducted at The Pennsylvania State University showed that the bilayer configuration behaved efficiently with modest levels of steel plate reinforcement (as low as 2 percent steel, by volume). The bilayer reinforced stressed timber deck configuration offers a sturdy and economical design, well suited for short and medium spans and ideal for low-volume roads.

Load testing of a 12.2-m (40-ft) bilayer deck (Butt Pattern C, Figure 1c) was conducted (3), which is the focus of this paper. Unlike the “ideal” bilayer configuration in which timbers with unequal widths are stacked vertically and alternated,
the bilayer prototype constructed for this study had a gap in the midheight of the cross section (Figure 2) because of the limited resources at the time of the study.

**BILAYER DECK CONFIGURATION AND CONSTRUCTION**

**Butt Pattern Comparison**

The bilayer butt pattern, or Butt Pattern C, was designed to minimize timber lengths and disperse butt joints as well as possible (Figures 1c and 2). The four-row pattern was repeated eight times across the width of the bilayer prototype deck for a total of 32 longitudinal rows of wood members. Bilayer deck timbers ranged in length from 1.8 m to 3.4 m (6 ft 0 in. to 11 ft 4 in.) and were 51 x 165 mm (2 x 6 1/2 in.) in cross section.

Whereas Butt Pattern B had 50 percent timber absent at any transverse row of butt joints (Figure 1b) with each butt joint encompassing the entire depth of the deck, Butt Pattern C was designed such that only 25 percent timber was absent at any row of butt joints. In addition, the butt pattern arrangement of the upper half of the deck was shifted relative to the lower half of the deck in Butt Pattern C to ensure that no butt joint continued through the full depth of the deck. Butt Pattern A possessed the most conservative design, having the fewest butt joints and, subsequently, the longest timber lengths (Figure 1a).

**Construction of Bilayer Prototype Deck**

A mix of unseasoned, rough-cut, No. 2 mixed red and white oak was requested for the bilayer prototype deck (Figure 2) to be consistent with previous studies. Although the authors recommend that timber widths be controlled to within 3.2 mm (1/8 in.) for reinforced stressed decks (4), bilayer prototype timbers varied in thickness by as much as 6.4 mm (1/4 in.).

Given the other inconsistencies present (variation in width, timbers not perfectly straight or true), it is reasonable to conclude that timber used in the prototype deck was fairly representative of rough-cut timber used in practice. Because testing was performed indoors, timbers were not treated with creosote. The 9.5-mm x 356-mm x 12.2-m (3/8-in. x 14-in. x 40-ft), high-strength, corrosion-resistant (ASTM A588) steel sandwich plates used in Butt Patterns A and B were used for Butt Pattern C.

Because the bilayer prototype was constructed and tested as part of an unfunded master of science project, several concessions were made in the deck design. The most notable is the use of a middepth material gap (Figure 3) to eliminate costs associated with fabrication of holes in timbers. With the absence of holes, the gap allowed stressing rods [Dywidag 25 mm (1 in.), high-strength steel threadbars, \( F_u = 1.03 \text{ MPa (150 ksi)} \)] to pass transversely through the deck. As a result, spacers were needed to support the upper deck approximately 32 mm (1 1/4 in.) above the lower deck. Although butting bilayer deck halves together will certainly be the chosen standard in practice, the chosen configuration (Figure 3) represents a "worst case" design for a bilayer deck, and measures of deck efficiency are somewhat conservative.

With bilayer deck timbers and stressing rods in place, bearing plates, anchor plates, and conical nuts were placed on each end of the rods (Figure 3). A hollow-core jack was used to slowly pull the loose deck timbers and steel plates together. Care was taken not to apply any prestress force until all gaps between laminates were removed. The stressing sequence found to minimize distortion of deck shape agrees with the suggested procedure in the *Quality Assurance and Inspection Manual for Timber Bridges* (5), which involves stressing midspan rods first and working toward outer rods.

**TESTING PROCEDURE**

The 12.8-m (41-ft 11-in.) bilayer deck rested on timber sills 305 x 305 mm (12 x 12 in.), having a span length of 12.2 m (40 ft 0 in.) from center to center of bearing areas. A military loading arrangement was used with jacks (servohy-
draulic (Figure 2) was used to transfer the load to the deck. Bridge
deflections were read to the nearest 1.5 mm (0.005 ft, or about
\( \frac{1}{32} \) in.) using a rotating laser (EGL Beam Machine) with an
automatic leveling base and a level rod. Deflections were
taken on each timber and steel laminate across the width of
the bridge at midspan.

The bilayer deck was tested with 15, 11, 7, 5, 3, and 2 steel
sandwich plates, and strain gauges were mounted on the top
and bottom of 5 steel sandwich plates as indicated (Figure 4).
Tests were performed at three prestress levels for each steel
plate reinforcement level. The chosen prestress levels were
356, 222, and 89 kN (80, 50, and 20 kips) per rod, corre-
sponding to prestress pressures of 1011, 632, and 253 kPa
(146.8, 91.8, and 36.7 psi).

Before each test, the deck was stressed to the required
prestress pressure and a static shakedown was applied. The
typical load sequence was 26.7, 53.4, 80.1, and 106.8 kN (6.0,
12.0, 18.0, and 24.0 kips) per jack; however, for stiffer decks,
a load sequence of 33.4, 66.8, 100.1, and 133.5 kN (7.5, 15.0,
22.5, and 30.0 kips) was often used. Conversely, for less heav­
ily reinforced decks, the load sequence was often modified or
cut short to keep materials in the elastic range. Loads were
applied quickly (about 5 sec between load levels) and deflec­
tions measured on all laminates in about 2 min. Because of
the presence of steel reinforcement plates, time-dependent
deflections due to creep in timber were found to be minimal.
In addition to the load deflection tests, a dynamic shake was
performed to measure interlaminate slip due to impact loads.

LOAD-DEFLECTION RESULTS FOR BILAYER DECK

Deflection often controls the design of stressed timber bridges,
so accurate prediction of deck stiffness is important. The most
significant variables that affect stiffness of stressed timber
bridges are longitudinal and transverse MOE of deck timbers,
quantity of steel sandwich plate reinforcement, degree of
transverse prestressing, frequency and location of butt joints,
and moisture content of deck timbers. Because of obvious
limitations in studying some of these effects, only the most
influential factors were examined. Thus, for testing of Butt
Patterns A, B, and C, the two chosen variables were quantity
of steel sandwich plate reinforcement and degree of transverse
prestressing pressure.

The master stiffness curves for the bilayer prototype deck
(for the six reinforcement and three prestress levels) are based
on average centerline deflections (Figure 5). Comparisons
between average and maximum centerline deflections are pre­
sented later in this section. Because the prestress pressure
induced from 222 kN (50 kips) force per rod is closest to a
typical prestress level maintained in the field, best-fit straight

![Figure 3: Cross section of bilayer prototype deck.](image)

![Figure 4: Steel plate locations in bilayer prototype deck.](image)

![Figure 5: Load-deflection results for all reinforcement and prestress arrangements (Butt Pattern C).](image)
lines were drawn through data points corresponding to that prestress level (Figure 5). Deck stiffness is defined as the slope of load-deflection curves in the elastic range and has units of force per length. The obvious trends seen are that stiffness is very dependent on steel plate reinforcement level and is somewhat dependent on prestress level. A quantitative measure of this difference is presented hereafter.

To study the effects of prestress pressure on deck stiffness, stiffnesses were computed for all three prestress pressures (Table 2) using linear regression of load deflection data. The linear model fit data well, with $r^2$ values ranging between 0.992 and 0.999, a finding consistent with other studies of stressed timber decks with steel sandwich plates. This result suggests that materials were kept within the elastic range. Another regression was made and stiffnesses calculated for the collection of load-deflection data of all three prestress levels within each reinforcement level. These values, displayed in rows labeled “all” (Table 2), represent an approximate mean stiffness value for a particular reinforcement level and were used to study the effect of prestress level on stiffness.

Stiffnesses for decks tested with 356 kN (80 kips) prestress force per rod were between 0.9 and 3.2 percent higher than the respective averages for “all” reinforcement levels (excluding two plate results). Stiffnesses for decks tested with

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<th>NUMBER OF STEEL PLATES</th>
<th>PRESTRESS FORCE PER ROD (kN)</th>
<th>K (kN/m) BASED ON AVERAGE CENTERLINE DEFLECTIONS</th>
<th>PERCENT DIFFERENCE BETWEEN PRESTRESS LEVELS</th>
<th>PERCENT DIFFERENCE BETWEEN REINFORCEMENT LEVELS</th>
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\textsuperscript{a} Percent difference of stiffness, K, in each plate category from the average (see note 'c' below) of that category.

\textsuperscript{b} Percent difference of stiffness, K, for 222 kN force per rod for all plate categories compared to a reference stiffness at 7 plates (14.1 kN/m).

\textsuperscript{c} Linear regression through data from all three prestress levels to obtain an approximate average.

\textsuperscript{d} Values for deck configuration with 2 plates may be representative of prototype only. Bilayers with similar reinforcement but without mid-height material gap may experience higher stiffness values.
222 kN (50 kips) force per rod were found to be within 0.7 percent of the average when containing between 3 and 11 plates. For decks tested with 89 kN (20 kips) force per rod, stiffnesses were found to be between 4.0 and 12.2 percent less than the average values when containing between 3 and 15 plates. Thus, prestress level has a small but significant effect on stiffness values, especially for lower prestress levels. The increase in average centerline deflections as prestress level is decreased indicates that a reduction of transverse stiffness has occurred in the structure, as described in the following section.

**Average Versus Maximum Centerline Deflection**

In the preceding sections, all stiffness values used in comparisons were based on linear regression through data points representing load versus average centerline deflection. Although this is a legitimate way to represent average stiffness values for the prototype, averaging of centerline deflections can mask the transverse deflection patterns of the platelike deck system. An accurate measure of stiffness based on maximum centerline deflections is important because load tests performed in the field on recently built structures presently involve placing resultant loads (triaxle trucks) at the centerline of decks and measuring deflections at many points along their widths to obtain the maximum deflection.

For the typical transverse centerline deflection pattern shown (Figure 6), note that the deflection profile is "choppy" because of the resolution of the measuring system used [1.5 mm (1/16 in.)]. Also, recall that a loading block 305 mm (12 in.) wide was used to deliver loads to the test deck 1.78 m (70 in.) wide. The linear deformation pattern seen in the middle of each plot is an effect of measuring on the front and back of the loading block 305 mm (12 in.) wide and interpolating to obtain deflections between these points. To help visualize the overall transverse deflection trends, a polynomial of the second degree was fit through data.

It is obvious that transverse stiffness of the deck is very dependent on prestress level. In no case was interlaminate slip detected, however, even at the lowest prestress level, under static and dynamic loading. In general, maximum centerline deflections are only about 5 percent larger than average centerline deflections for the half-lane prototype deck.

**Calculated and Measured Stresses in Wood and Steel**

Because timber stresses were not physically measured, they were estimated using simple beam theory for the various deck arrangements and plotted versus percent steel (Figure 7). In Ritter's treatment of longitudinal stressed-laminated deck design (6), allowable bending stress \( F_b \) is calculated by adjusting tabulated single-member allowable bending stress for moisture \( C_M \), and load sharing \( C_{LS} \), and not for size factor \( C_F \). Taking the base value for allowable bending stress in the supplement to the 1991 National Design Specification for Wood Construction (NDS) tables (7) and making the proper adjustments gives

\[
F_b = F_b C_M C_{LS} = (5.51 \text{ MPa})(1.0)(1.5)
\]
\[
= 8.27 \text{ MPa} (1,200 \text{ psi})
\]

Note that the 1991 NDS specifies that when \( (F_b) (C_F) \leq 7.9 \text{ MPa} (1,150 \text{ psi}) \), \( C_M = 1.0 \) is used. Even if \( C_F = 1.3 \) was applied, \((5.51)(1.3) = 7.2 \text{ MPa} (1,040 \text{ psi}) \leq 7.9 \text{ MPa} (1,150 \text{ psi}) \), thus \( C_M = 1.0 \) was used. By comparing this allowable bending stress value with plotted data (Figure 7), it appears that bending stresses in wood are critical for the loads considered. However, optimum design studies (4) indicate that the optimum deck thickness for a multilayered deck having a 12.2-m (40-ft) span length is 457 mm (18 in.), considerably deeper than the 368-mm (14.5-in.) bilayer deck. In fact, it was found that when generating optimum design curves, live load deflections were almost always the controlling design criteria, not timber stresses.

Consider the plot of calculated and average measured steel stresses versus percent steel in the deck (Figure 8). The al-

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**FIGURE 6** Transverse centerline deflection pattern for bilayer deck arrangement with three steel plates (Butt Pattern C).

**FIGURE 7** Calculated timber stresses versus percent steel in bilayer prototype deck (Butt Pattern C).
lowable design stress in ASTM A588 steel \( F_y = 345 \text{ MPa} \) (50 ksi) according to AASHTO's Standard Specifications for Highway Bridges (8) is
\[
F_{\text{all}} = 0.55 (345 \text{ MPa}) = 189 \text{ MPa} (27.5 \text{ ksi})
\]
For the bilayer prototype, steel stresses approach this limit with seven reinforcement plates in the deck (3.75 percent steel) and \( P = 107 \text{ kN} \) (24 kips). In all cases, measured steel stresses are lower than calculated steel stresses. One possible explanation for this consistent difference might be that the modular ratio used in determining calculated steel stresses is based on \( E_{\text{APP(WOOD)}} \) which assumes that timber properties are constant throughout the entire deck. In reality, the deck has lower stiffness at butt joint locations and higher stiffness where strain gauges were located [at midspan, where wood contained no butt joints for several feet in either direction (Figure 1c)]. Again, note that when considering optimum deck thickness of 457 mm (18 in.), stresses would be considerably lower and that live load deflections almost always govern design thickness.

**COMPARISON OF BUTT PATTERNS A, B, AND C**

Relative efficiencies of reinforced stressed timber decks were compared using longitudinal flexural rigidity ratio (LFRR). The LFRR is the ratio of apparent EI, or internal stiffness of wood and steel for a given deck arrangement, over the base EI, or the internal stiffness of a solid wood deck made up of clear wood only. This dimensionless quantity is not biased toward section properties and wood MOE, making it an ideal choice for comparing efficiencies of Butt Patterns A, B, and C.

A plot of LFRR versus percent steel (Figure 9) for the single-layered decks (Butt Patterns A and B) and the bilayer deck (Butt Pattern C) shows the following:

1. Butt Pattern A is most efficient, followed by Butt Patterns B and C, respectively. This was expected because Pat-

tern B had twice as many butt joints as A, and Butt Pattern C had a large number of butt joints combined with a midheight material gap.

2. LFRR curves bend slightly upward as the percentage of steel is increased. Although increasing steel content leads toward linearly increasing stiffness, the wood contributes more to overall stiffness of the deck as the amount of steel is increased (even though the quantity of wood in the deck is always constant). The wood contribution increases with steel content because of the increased splicing action provided by the addition of steel plates. Such splicing action plays a major role in the utilization of small lengths and narrow widths of timbers.

The nonlinear drop in stiffness between three and two plates for Butt Pattern C indicates that a breakdown has occurred in the ability of the bilayer to act as an efficient composite deck at this very low steel content level. Because of the midheight gap in material of the bilayer prototype cross section (Figure 3), the upper half of the two-layered configuration was free to deflect more than the lower half in the two-plate (Figure 4a) configuration (1.10 percent steel content). Because no measurements were taken to prove that the phenomenon previously described actually occurred (for instance, deflection measurements taken on the top and bottom of the deck), only limited inferences can be made concerning the presence of the nonlinear drop in stiffness. However, it is likely that a typical bilayer configuration would perform better at this low reinforcement level.

**CONCLUSIONS**

Prestress level was found to be an important factor in bilayer deck stiffness. Despite the increasing loss of longitudinal and transverse stiffness detected in examining transverse centerline deflection patterns for the three prestress levels, there was no evidence of slippage between adjacent laminates for even the lowest prestress level. This result suggests that although keeping prestress levels near the target level is im-
important in maintaining design stiffness characteristics, pre‐
tress loss is, in general, a forgiving and detectable phenomenon
rather than a catastrophic one.

The bilayer prototype deck possessed a 38-mm (1½-in.)
midheight gap between upper and lower layers of wood. De‐
spite this undesirable design detail, the bilayer deck showed
considerable stiffness and efficiency at all but the lowest re‐
inforcement level, which involved using only two steel plates
in the half-lane deck. To allow lower reinforcement levels
such as these, use of a variety of timber thicknesses to make
up the bilayer cross section would be helpful.

The relative efficiencies of Butt Patterns A, B, and C were
compared by examining the LFRR versus percentage of steel
in deck. For LFRR values of 1.0, Patterns A, B, and C need
approximately 0.5, 1.5, and 1.8 percent steel content, re‐
spectively. To have LFRR values of 2.0, Patterns A, B, and
C need 5.0, 5.5, and 6.5 percent steel content, respectively.

This study clearly shows that the bilayer configuration is
not only possible but necessary in making longer stressed
timber spans economically feasible. However, further study
in this area is needed to investigate

1. Behavior of multilane decks and decks of varied thickness,
2. Effects of preservative treatment on interlaminate fric‐
tion and deck behavior,
3. Behavior of decks constructed with other hardwood
species,
4. Ultimate capacity of reinforced decks, and
5. Effects of moisture content, steel channels on deck sides,
and so forth.

Steel sandwich plates in stressed timber bridge decks reduce
camber loss due to creep in timbers and enable longer spans.
The presence of reinforcement plates also permits the use of
both shorter and lower-grade timber, thus significantly low‐
ering timber costs. Additional timber savings are possible with
bilayer or multilayer decks, which use timber from smaller
trees.

Timber bridges have always been admired for their simple
and natural beauty and for using a renewable, widely available
resource. Now, with modern designs such as the bilayer rein‐
forced stressed deck, timber bridges are becoming a viable
alternative for short- and medium-span bridges.

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Development and Field Testing of the Camp Arrowhead Modular Stress-Laminated T-System Timber Bridge

Julio F. Davalos, Hani A. Salim, and Barry Dickson

The development and field testing of a new modular stress-laminated T-system timber bridge, which is expected to become cost-competitive with precast concrete bridges, is presented. The new modular design can increase the quality of the product and decrease fabrication and installation efforts. The analysis methods used to predict the bridge response include orthotropic finite element (FE) modeling, a macrosolution for a stiffened plate, and a design method based on the FE modeling and macroanalyses. Details of the fabrication and installation procedures are presented. The Camp Arrowhead bridge is tested under a 231-kN (52-kip) loaded truck, and the measured and predicted responses are compared and discussed.

Modern timber bridges, recently built in West Virginia, consist of hardwood stress-laminated decks combined with softwood glued-laminated beams compressed together by high-strength steel bars to form T- and Box-systems. Current design codes do not provide standards for stress-laminated T-system timber bridges. Therefore, the Constructed Facilities Center (CFC) at West Virginia University (WVU) has developed design procedures, called the WVU method, for the design of these systems (1). The method has been adopted by the West Virginia Department of Transportation (WVDOT). However, higher-than-expected costs and inefficient construction practices have brought about a need for innovative design for stress-laminated T-system timber bridges. In response, a modular stress-laminated T-system timber bridge, which is expected to become competitive with precast concrete bridges, has been developed.

Modular T-system timber bridges developed in West Virginia consist of cells or modules, each about 122 to 152.4 cm (4 to 5 ft) wide and extending the full length of the bridge. Two glued-laminated timber (glulam) beams are transversely stress-laminated to approximately 30 deck lumber planks [3.8 cm (1.5 in.) thick] to form a single module. The depth of the beams depends on the bridge span and loading. Figures 1 and 2 show the cross section details of the Camp Arrowhead timber bridge, which was designed using the WVU method.

This paper presents an overview of the design and details of the construction, testing, and response evaluation of the two-lane, 18.9-m (62-ft) span Camp Arrowhead bridge built in Cabell County, West Virginia, in 1992. Response predictions are obtained by finite element (FE) analysis, a macrosolution, and the simplified WVU design method. The bridge is tested using a 231-kN (52-kip) double-axle truck, and deflections and strains are recorded and compared with predicted values.

ANALYSIS AND DESIGN OVERVIEW

The design of stress-laminated T-system timber bridges is based on a macroflexibility solution of a stiffened orthotropic plate. A one-term approximation of this solution is used to define a wheel load distribution factor, which reduces the design of the structure to the design of a T-beam section. To analyze this T-beam section using beam theory, an effective flange width is computed from expressions obtained from a parametric FE analysis. The overview presented in this section describes the longitudinal global response, the transverse local effects in a deck section between two adjacent stringers, and design considerations.

Longitudinal Global Response

A generalized deflection function of a simply supported orthotropic plate stiffened by longitudinal stringers is obtained by a macrosolution (1,2). Using a first-term approximation, a transverse wheel load distribution factor \( W_r \) is defined as the ratio of the interactive forces acting on a stringer to the sum of interactive forces acting on all stringers (3). Using this concept, the maximum wheel load distribution factor for a symmetric load on an interior stringer can be written as

\[
W_r = \frac{1 + C_0}{(n + 1)C_0 + \frac{2}{\pi} n}
\]  

(1)

where

- \( C_0 \) = edge deflection coefficient = \( \frac{b}{\pi D_s\left(1 + 8\gamma^2\right)} \),
- \( D_s \) = transverse bending stiffness of deck,
- \( D_s \) = bending stiffness of composite edge stringer,
- \( \gamma \) = aspect ratio = bridge width/bridge length, and
- \( n \) = number of stringer spacings.

Equation 1 is presented to illustrate that the wheel load distribution factor \( W_r \) accounts for the orthotropic property of the system and simulates the portion of the actual truck loading carried by an interior stringer. Therefore, the defi-
nition of \( W_f \) reduces the design of the system to the design of a composite T-beam. However, since the normal stress distribution in the flanges of the T-beam is nonlinear (Figure 3), we define an equivalent effective flange width over which the normal stresses can be assumed to be constant consistent with beam theory. The effective overhanging flange width \( B_E \) is computed from

\[
B_E = 0.4586 + \frac{1}{198} \left( \frac{L}{B} \right) \left( \frac{D}{t} \right) \left( \frac{E_s}{E_d} \right)
\]

where \( L \) is the bridge span, \( E_s \) and \( E_d \) are, respectively, the longitudinal elastic moduli of the stringer and the deck, and \( D \) and \( t \) are defined in Figure 3. Then, \( b_e \leq S \).

In the WVU design method, the wheel load distribution factor (Equation 1) and the effective flange width (Equation 2) permit the designer to isolate a deck-and-beam portion of the bridge (Figure 3) and to design it as a T-beam. The T-beam is loaded at the center by an equivalent concentrated load \( P_d \) that produces a maximum moment equal to the maximum AASHTO lane moment (4), modified for wheel load distribution and number of lanes. Using Equation 1, the maximum live load moment, \( M_{LL} \), and deflection, \( \delta_{LL} \), for an interior stringer are computed from (I)

\[
M_{LL} = \frac{P_d L}{4} \quad (3)
\]

\[
\delta_{LL} = \frac{P_d L^3}{48 E I_c} \quad (4)
\]

where

\[
P_d = (\text{AASHTO lane moment}) \left( \frac{4}{7} \right) (N_s)(W_f).
\]

\( N_s \) = number of lanes,

\( E_s \) = modulus of elasticity of the stringer, and

\( I_c \) = transformed composite moment of inertia of the effective T-section (Figure 3).

To analyze the global longitudinal response of the bridge, the total normal stresses and longitudinal deflections are computed from beam theory using the properties of the transformed T-beam section.

**Transverse Local Response**

For a trial deck thickness, the spacing of the stringers \( S \) is designed by computing the maximum transverse local deflection \( \delta_{max} \) and stress \( \sigma_{max} \) (local effects) directly under a wheel load applied at the midspan of a deck section between stringers. The limits on local deflection can be based on human response and pavement cracking considerations. The local effects are computed from (I)

\[
\delta_{max} = \frac{PS^3}{4 \alpha E T^4} \quad \sigma_{max} = \frac{3PS}{2 \beta T^3}
\]

where

\[
\alpha = -10.9 + 7.8 \left( \frac{S}{t} \right) + 0.27 \left( \frac{E_k}{E_T} \right),
\]

\[
\beta = 3.0 + 3.1 \left( \frac{S}{t} \right) + 0.152 \left( \frac{E_k}{E_T} \right)
\]
The design procedure described in this section applies to BRIDGE strength of the glulam stringers and the maximum local deflection of a deck section (relative to the stringers) control. The exterior stringer is designed for asymmetric loading, as shown required depth of the stringers is 114 cm (45 in.).

The modular construction technique used for this bridge is asymmetric loading, as shown required depth of the stringers is 114 cm (45 in.). and of the interior stringers is 12.7 cm (5 in.); the interior stringer is designed for symmetric AASHTO truck loading, as described previously, and the exterior stringer is designed for asymmetric loading, as shown elsewhere (7).

The configuration, construction, and testing of the Camp Arrowhead bridge are presented next.

**BRIDGE CONFIGURATION**

The design is for a bridge of two lanes, 18.9-m (62-ft) span, and 7.24-m (23.75-ft) out-to-out width subjected to an AASHTO HS-20 truck loading. The bridge consists of five cells or modules. The interior modules are 140 cm (55 in.) out-to-out, and the exterior modules are 152.7 cm (60 in.) out-to-out (Figures 1 and 2). The deck is built with 3.8- x 23-cm (1.5- x 9-in.) northern red oak lumber (No. 2 grade), and the glulam stringers are 24F-V3, Southern Pine/Southern Pine (5). The width of the exterior stringers is 25.4 cm (10 in.) and of the interior stringers is 12.7 cm (5 in.); the interior stringers are placed side by side to form a beam 25.4 cm (10 in.) wide. On the basis of the WVU design method, the required depth of the stringers is 114 cm (45 in.).

**BRIDGE CONSTRUCTION**

1. Each module consists of two full-length glulam beams and approximately 30 deck planks 3.8 cm (1.5 in.) thick (Figure 1).
2. Stressing bars are located on 61-cm (2-ft) centers with an additional set of fabrication bars located at 183-cm (6-ft) centers.
3. The fabrication bars have anchor plates that are inserted into daps cut into the glulam beams 12.7 cm (5 in.) wide, so that no hardware protrudes past the face of the beam (Figure 2).
4. The bars on 61-cm (2-ft) centers and the fabrication bars are tensioned three times at the fabrication shop, which completes the process required to minimize bar force losses during the expected life of the bridge.
5. Curbs and guiderail components are added to the exterior modules at the fabrication shop.
6. The modules are shipped to the bridge site, and the bars at 61-cm (2-ft) centers are removed, whereas the bars on 183-cm (6-ft) centers are left in place.
7. The modules are lifted into position on the prepared bridge seats.
8. The full-length stressing bars are pushed through the vacated holes on 61-cm (2-ft) centers, and the anchorage hardware is installed to tension the bars one time only.
9. Finally, the bridge is fastened to the abutments, an asphalt overlay is applied, and the bridge is ready for vehicular traffic.

The time required for the entire fabrication process still takes approximately 8 weeks, but the new modular process allows the on-site work to be done in 1 day rather than the 3 or 4 weeks required of our previous construction methods.

**BRIDGE TESTING**

The monitoring program of the Camp Arrowhead bridge includes measurements of bar force levels, moisture levels, live load deflections, and strains. The Camp Arrowhead bridge has been in service for only 4 months, but it appears that the bar forces are stable. The average prestress level of two of the prestressing bars is between 689 and 345 kPa (100 and 50 psi), which is considered acceptable. The average moisture content (MC) level in the bridge measured over a period of 4 months is between 17 and 21 percent; the assumed MC level in design is greater than or equal to 19 percent. The 231-kN (52-kip) loaded truck used to test the bridge was less than the design AASHTO HS-20 loading. Therefore, the actual truck load was used in the analysis to compare the predictions with the experimental field results. The location of the loaded truck, deflection measurements, and strain measurements are explained next.

**Location of the Loaded Truck**

A double-axle, 231-kN (52-kip) loaded truck was placed over the bridge, and the response of the superstructure was tested for three load conditions:

1. The truck was placed on the downstream lane of the bridge, facing the traffic direction (north); the center of gravity of the truck coincided with the midspan of the bridge.
2. In a similar manner, the truck was placed facing south on the upstream lane of the bridge.
3. The interior rear wheel of the truck was placed right at the midspan of the deck section of the middle module of the bridge (see Figure 4, which shows the deck section between Stringers 3 and 4 denoted in Figure 1).

To measure the global live-load deflections with an engineer's level, sight rods were attached to the bottom of each of the six glulam beams. The rods were placed at quarter points along the span, which resulted in a total of 18 elevation reading points. To measure the local deflections in the middle deck section, relative to Stringers 3 and 4, two dial gauges were mounted on a steel angle bolted to the stringers (Figure 4).

**Deflection Measurements**

To measure the global live-load deflections with an engineer’s level, sight rods were attached to the bottom of each of the six glulam beams. The rods were placed at quarter points along the span, which resulted in a total of 18 elevation reading points. To measure the local deflections in the middle deck section, relative to Stringers 3 and 4, two dial gauges were mounted on a steel angle bolted to the stringers (Figure 4).

**Strain Measurements**

In general, to measure strains in wood with bonded strain gauges is a difficult task (13). Moreover, bonded gauges cannot be used in creosote-treated wood. Therefore, to measure strains in the Camp Arrowhead bridge, we used laboratory-built, clip-on strain transducers (14). Stringers 3 and 4 were each instrumented at the midspan with two clip gauges, which were placed on the bottom faces of the beams. Similarly, a clip gauge was placed transversely at the bottom face of the middle deck section (see Figure 4) to measure the transverse strain at the midspan of the deck.

**BRIDGE RESPONSE EVALUATION**

For comparative purposes, the Camp Arrowhead bridge is analyzed by an FE method, a macrosolution for a stiffened plate (2), and the WVU design method. A special FE formulation (15) modified for T-systems (1) is used in the analysis, and the deck is modeled with nine-node, orthotropic shell elements that include shear deformations. The beams are modeled with three-node, transversely isotropic (16), three-dimensional beam elements that include shear deformations. The comparisons of the global and local deflections and stresses are discussed in this section.

The longitudinal deflection profile of the middle stringer is shown in Figure 5. The FE maximum deflection prediction is within 2 percent of the experimental results, and the WVU design predictions are within 22 percent of the experimental results. The macrosolution prediction for a one-term approximation is within 13 percent of the experimental results. The transverse deflection profile is shown in Figure 6. Considering that the accuracy of the deflection measurements in the field is ±1.6 mm (± 16 in.), the WVU design predictions are reasonably accurate.

The local transverse deflection in the deck section was measured under the interior rear wheel of the truck (see Figure 4). In the FE analysis, the deflections due to the interior and exterior rear wheel loads, at the location of the interior wheel, were computed separately. In the WVU method, only

![FIGURE 4 Measurement of beam strains and local effects.](image-url)
the transverse deflection exactly underneath the wheel load can be computed; therefore, the FE solution for the deflection due to the exterior rear wheel [1.31 m (4.3 ft) away] is added to the WVU method prediction and compared with the measured deflection. The same approach was used to compute the local transverse stresses. The experimental strains are converted to stresses by multiplying them by the transverse modulus of elasticity of the deck, which is assumed to be 172.25 MPa (25,000 psi) for a transverse prestress level of 345 kPa (50 psi) (8). The results, summarized in Table 1, show that the deflection predictions compare well with the field measurements. The predicted transverse stresses are 17 percent higher than the field data; this percent difference for structural timber is considered reasonable and acceptable (13).

The longitudinal global strains in the bottom surfaces of Stringers 3 and 4 were measured with clip-on gauges. The strains were converted to stresses by assuming a modulus of elasticity of 13.78 GPa (2.0 x 10^6 psi) for the tension laminae of the stringers. The global stresses and deflections for Stringers 3 and 4 are compared in Table 2. The WVU design deflection prediction is 12 percent higher than the FE prediction and only 3 percent higher than the field measurement. The FE stress predictions are only 8 percent higher than the measured values. However, the WVU design stresses are 13 percent higher than the FE solution and 22 percent higher than the measured value. Since the design method is expected to be conservative, the stress predictions are reasonably acceptable.

A visual inspection of the Camp Arrowhead bridge and an assessment of the MC levels, bar tension levels, and load response of the bridge indicate that the overall performance of the bridge is satisfactory and within the design expectations.

CONCLUSIONS

The development and load testing of a new stress-laminated, modular, T-system timber bridge are presented. This modular

<table>
<thead>
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<th>TABLE 1 Local Response in Deck Section Between Stringers 3 and 4</th>
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<tr>
<td>deflection (cm)</td>
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<td>stresses (kPa)</td>
</tr>
</tbody>
</table>

^From FE analysis

Note: 1 in = 2.54 cm; 1 psi = 6.89 kPa

<table>
<thead>
<tr>
<th>TABLE 2 Global Deflections and Stresses of Interior Stringer</th>
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<td>max. deflection (cm)</td>
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<td>max. stresses (MPa)</td>
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</tbody>
</table>

Note: 1 in = 2.54 cm; 1 psi = 0.00689 MPa
system reduces fabrication, transportation, and installation efforts and allows for better quality control. The WVU design method described in this paper, based on rigorous closed-form and FE analyses, is sufficiently simple and reasonably accurate to predict deflections and stresses of stress-laminated T-system timber bridges. The comparisons with the test results indicate that the WVU design method predicts quite well the response of the Camp Arrowhead bridge and can be used for the design of stress-laminated T-system timber bridges. The Camp Arrowhead bridge is performing well in relation to its original design.

ACKNOWLEDGMENT

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Hardwood Glued Laminated Timber Bridges

H. B. MANBECK, J. J. JANOWIAK, P. R. BLANKENHORN, AND P. LABOSKY

Design standards and specifications for 5.5- to 27.4-m (18- to 90-ft) clear span hardwood glued-laminated (glulam) highway bridges have been developed and are available from the Pennsylvania Department of Transportation. Resin systems, preservative treatment processes, laminating procedures, and key structural properties have been determined and incorporated into the standards for three commercially important hardwood species: northern red oak, red maple, and yellow poplar. The keys to successfully bonding the hardwoods are proper open assembly time and clamping pressure. Pressure treatment cycles to attain 160.2 to 192.2 kg/m² (10 to 12 pcf) retention of creosote in northern red oak, red maple, and yellow poplar glulam girders had no adverse effect on the flexural strength or stiffness.

Standard designs and specifications for hardwood glued-laminated (glulam) timber highway bridges have been developed at the Pennsylvania State University for the Pennsylvania Department of Transportation (1). The designs and specifications are similar to those for softwood glulam bridges (2,3) but are specifically developed for glulam bridges fabricated with northern red oak, red maple, and yellow poplar lumber. Development of the standards required identification, qualification, or development of resin systems and laminating processes for fabricating the glulam structural elements, preservative treatment processes, allowable design strengths and stiffnesses, and fastening systems. This paper summarizes the research forming the basis of the standards, the hardwood bridge standards, and the design and performance of a hardwood glulam bridge.

RESIN SYSTEMS AND LAMINATING PROCESSES

Three hardwood species with good potential for development of hardwood glulam timber bridges are northern red oak, red maple, and yellow poplar (4). Gluing and preservative treatment processes that satisfy quality assurance standards established by the American Institute of Timber Construction (5) and the American Wood Preservers' Association (6) were developed for each species.

A comprehensive discussion of the requirements for glued joints in hardwood glulam construction, selection and use of adhesives for hardwood glulams, selection and preparation of hardwood lumber for laminat, lay-up of hardwood laminated assemblies, adhesive edge and end joint connections, adhesive face lamination procedures, and clamping pressure is presented by Manbeck et al. (4). The recommendations in the report provide guidance to glulam fabricators and engineers when modifying procedures for quality assurance for hardwood glulam members. Only minor deviations from softwood manufacturing technology are needed for acceptable hardwood glulam timbers. Lamination procedures, which significantly influence the bonding process, must be properly adjusted for higher-density hardwoods. Higher-density substrates of red oak and red maple, in comparison to bonding of softwoods, require greater attention to lamination surface quality and applied clamp pressures. Laminators should have minimum difficulties in utilizing yellow poplar with existing manufacturing technology. Red oak and red maple will require some modification or refinement of existing softwood-based gluing practices. Hardwood glulam manufacture should not represent a significant added production expense or capital investment for most established lamination operations. Slightly higher production costs over softwood glulam should be anticipated until hardwood materials are available as standard-dimension lumber. Higher hardwood glulam costs are related in part to the loss in production efficiency when dealing with semiprocessed S2S hardwood lumber instead of standard finished nominal-sized lumber products. Costs should decrease as an infrastructure develops for nominal dimension and graded hardwood lumber. Production costs are increased because more costly resorcinol formaldehyde (RF) adhesives are used. However, most softwood operations using phenol resorcinol formaldehyde (PRF) resins can easily convert to RF face lamination adhesives. Capital investments by the laminator will be required if the glulam operation needs to update clamping assembly processes to achieve higher clamping pressures. The fabricator may also need to upgrade the planer to obtain more stringent surface quality requirements. Lamination procedures must be monitored for glulam manufacturers who have limited experience bonding higher-density hardwood lamination stock. Laminators with no experience must show evidence of their ability to conform with ANSI A190.1 (7).

Research also indicated that either vertical or horizontal finger-joint orientations with melamine adhesive are effective for end-joint fabrication meeting the AITC qualification criteria. Qualification data indicated that finger-joint performance was adequate for glulam manufacture with a 16.54-MPa (2,400-psi) or greater beam design value. Experimental results indicated that melamine formaldehyde (MF) was a viable adhesive for hardwood finger-joint assembly with good bonding performance. Testing has shown that MF formulations can provide acceptable performance for the three hardwoods even after exposure to high-moisture conditions.
Oil-borne preservative treatments, such as creosote and pentachlorophenol, are required for hardwood glulam members used in bridge applications. On the basis of criteria such as service life and availability of treating facilities, creosote was selected as the preservative for hardwood glulam bridge members. A comprehensive discussion of the requirements for preservative treatment of northern red oak, red maple, and yellow poplar glulam beams is presented by Manbeck et al. (4). The report includes discussions of preservative retention requirements and performance, treatment cycles, penetration of preservative, and glueline performance after treatment. The results of the treatment studies are as follows:

- Average weight creosote retention levels were 179.8 kg/m³ (11.2 pcf), 293.4 kg/m³ (18.3 pcf), and 257.2 kg/m³ (16.1 pcf) for northern red oak, red maple, and yellow poplar, respectively.
- Assay retention ranges were 121.8 to 181.0 kg/m³ (7.6 to 11.3 pcf), 309.2 to 406.9 kg/m³ (19.3 to 25.4 pcf), and 147.4 to 320.8 kg/m³ (9.2 to 18.9 pcf) for northern red oak, red maple, and yellow poplar, respectively.
- Minimum depths of penetration were 2.5 mm (0.1 in.) on the edge and 20.3 mm (0.8 in.) on the face for northern red oak, 27.9 mm (1.1 in.) on the edge and 25.4 mm (1.0 in.) on the face for red maple, and 7.6 mm (0.3 in.) on the edge and 10.2 mm (0.4 in.) on the face for yellow poplar.
- The effect of the preservative treatment cycle on glue bond performance, as measured by shear tests, was not significant.
- Northern red oak, red maple, and yellow poplar glulam beams may be treated with creosote to acceptable AWPA levels (6) in commercial operations. The treatment cycle will depend on the commercial operation and treatment facility. The creosote treatment does not adversely affect shear strength or percent wood failure. The three species will pass the cyclic delamination test with selected resins before creosote treatment provided that the gluelines in the untreated glulam beams are sound.

Posttreatment cycles have also been developed to minimize bleeding of preservative from treated glulam bridge members. Cycle details are described by Manbeck et al. (4).

ALLOWABLE DESIGN VALUES

Overview of Research Goals

Allowable design values (ADVs) were determined for northern red oak, red maple, and yellow poplar glulam beams loaded perpendicular to the plane of the laminations (bending about the x-axis in Figure 1a) and loaded parallel to the plane of the laminations (bending about the y-axis in Figure 1a). Allowable flexural strengths ($F_{w}$) and stiffness ($E$) were also experimentally obtained for one lamination lay-up of northern red oak and red maple. All other ADVs, including those for shear and bearing strength, were estimated and deduced from published values for the predominant grade of the species lumber used in the beam laminates (10). The girders and deck panels used in hardwood timber bridges are treated with creosote, after fabrication. To obtain the treatment retention levels required by AWPA (6), normal treatment pressures and temperatures had to be modified. Thus, it was necessary to determine whether the postfabrication treatment of hardwood glulam beams with creosote to AWPA retention levels adversely affected the strength or stiffness. An experiment was conducted to test the hypothesis that preservative treatment had no effect on the strength or stiffness of northern red oak, red maple, or yellow poplar glulam beams.

Research was also conducted to determine whether (a) the methods outlined in ASTM 3737 (8) to predict the flexural strength and stiffness of softwood glulam beams are applicable to hardwood glulam beams, (b) it is technologically feasible to design and fabricate hardwood glulam beams with $F_{w} = 16.5$ MPa (2,400 psi) and $E = 12.4$ GPa (1.8 x 10^6 psi), and (c) the volume reduction effect for hardwood glulam beams is the same as that defined for softwood glulams in the National Design Specification for Wood Construction (NDS) (9). This phase of the study focused on red maple and yellow poplar.

Combination A lay-ups, as defined in the NDS (9) and AITC-119 (10), were used for all the northern red oak studies (Figure 1a). Treatment effects for red maple and yellow poplar glulam beams were also evaluated using Combination A lay-ups. ADVs for red maple and yellow poplar were measured and predicted by ASTM 3737 (8) for the lay-ups shown in Figure 1b for red maple. Yellow poplar laminated lay-ups were similar, but not identical, to those in Figure 1b.

Summary of Results

Comprehensive discussions of the methods and results of the treatment effect and ADV research are included in several research reports and articles (4,11-13). The key results are as follows:

- Postfabrication treatment with creosote to retention levels specified by AWPA (6) did not adversely affect the flexural strength or the stiffness of northern red oak, red maple, and yellow poplar glulam beams.
- The dry-use flexural strength ($F_{w}$) of Combination A northern red oak glulam beams, as calculated from experimental MOR data from test beams, exceeded the values published in the NDS (9) for generic red oak. Calculated allowable flexural strength for 40 beams was 23.6 MPa (3,420 psi); the published value for red oak glulam (9) is 15.4 MPa (2,240 psi). An allowable value of 16.5 MPa (2,400 psi) is recommended for design.
- The dry-use stiffness ($E$) of Combination A northern red oak glulam beams, as calculated from experimental MOE data from test beams, exceeded the NDS (9) published value for generic red oak. The measured allowable stiffness was
13.1 GPa (1.9 x 10^6 psi); the published value for red oak (9)
is 11.0 GPa (1.6 x 10^6 psi). An allowable value of 12.4 GPa
(1.8 x 10^6 psi) is recommended for design.

- The allowable dry-use flexural strength ($F_{bx}$) of both red
maple and yellow poplar with lamination lay-ups similar to
those shown in Figure 1b both exceeded 16.5 MPa (2,400 psi)
and were satisfactorily predicted by the methods outlined in
ASTM 3737 (8).

- The allowable dry-use stiffness ($E_x$) of both red maple
and yellow poplar lamination lay-ups shown in Figure 1b equaled
12.4 GPa (1.8 x 10^6 psi) and was satisfactorily predicted by
the methods outlined in ASTM 3737 (8).

- ASTM 3737 (8) can be used to design red maple and
yellow poplar glulam beam cross sections with specified strength
and stiffness.

- The volume effect for red maple and yellow poplar glulam
beams is similar to that for softwood glulam beams. That is,
flexural strength ($F_{bx}$) declines as beam volume increases.
Stiffness ($E_x$) is unaffected by volume. The volume reduction
factor ($C_v$) is defined by Equation 1 for both red maple and
yellow poplar beams:

$$C_v = \left(\frac{b_0}{b}\right)^x \left(\frac{d_0}{d}\right)^z \left(\frac{L_0}{L}\right)^y$$  \hspace{1cm} (1)

where

- $b_0 = 130$ mm (5.125 in.),
- $b = \text{cross section width (mm)},$
- $d_0 = 305$ mm (12 in.),
- $d = \text{cross section depth (mm)},$
- $L_0 = 638$ m (21 ft),
- $L = \text{beam length (m)},$ and
- $x = y = z = 0.071$ (red maple and yellow poplar).

- Recommended dry-use ADVs for each species are sum-
marized in Table 1 for the conditions specified in the footnotes
to the table.
The bridge was completed and opened for traffic in November 1991. A demonstration bridge project has been underway in Pennsylvania for the past several years. The goals of this effort are to design, construct, and monitor hardwood timber highway bridges throughout the state, thus demonstrating the suitability of hardwoods for structural components in highway bridges. To date several hardwood transverse-stressed-longitudinal-deck, both unreinforced and steel-plate reinforced, demonstration bridges have been completed. At least three of the proposed demonstration bridges are to be hardwood glulam bridges, one each of northern red oak, red maple, and yellow poplar. The objective of the remainder of this section is to summarize the design and field performance of a northern red oak hardwood glulam demonstration highway bridge that was completed and opened for traffic in November 1991. Design details are further described by Manbeck et al. (15).

### NORTHERN RED OAK DEMONSTRATION BRIDGE

A demonstration bridge project has been underway in Pennsylvania for the past several years. The goals of this effort are to design, construct, and monitor hardwood timber highway bridges throughout the state, thus demonstrating the suitability of hardwoods for structural components in highway bridges. To date several hardwood transverse-stressed-longitudinal-deck, both unreinforced and steel-plate reinforced, demonstration bridges have been completed. At least three of the proposed demonstration bridges are to be hardwood glulam bridges, one each of northern red oak, red maple, and yellow poplar. The objective of the remainder of this section is to summarize the design and field performance of a northern red oak hardwood glulam demonstration highway bridge that was completed and opened for traffic in November 1991. Design details are further described by Manbeck et al. (15).

### Project Team

The project was a cooperative effort of several organizations under the leadership of a Penn State University research team. The Penn State team was responsible for all quality control matters and specifications related to wood procurement, processing, grading, and fabrication. Gwin Dobson and Forman, Inc., of State College, Pennsylvania, designed the substructure and superstructure and supervised construction; Unadilla Laminated Products, Inc., of Sidney, New York, fabricated the glued laminated structural members and fastener hardware; Koppers, Inc., of Muncy, Pennsylvania, treated the glulam members, and Kamtro Construction of Osceola Mills, Pennsylvania, constructed the bridge. The bridge owner is Ferguson Township in Centre County, Pennsylvania.

### Design Requirements and Procedures

A northern red oak glulam girders and deck was designed to replace a 44-year-old reinforced concrete tee beam bridge with a 107-kN (12-ton) rating on Township Road T-330 in Ferguson Township in Centre County, Pennsylvania. The bridge superstructure was erected on the existing stone abutments. The bridge skew, at 45 degrees, was severe.

The design requirements for the bridge were as follows: loads, HS25 or ML80 live load; deflections, live load deflection less than span/500; materials, all superstructure, railings, and parapets to be glulam northern red oak; clear span between centerline of abutments, 10.69 m (35 ft ½ in.); and overall deck width, 8.54 m (28 ft).

All structural components were designed in accordance with the 1986 edition of the National Design Specifications for Wood Construction (16), the 1988 edition of the Supplement to the National Design Specification (17), the AASHTO Standard Specifications for Highway Bridges (18), and PennDOT’s Design Manual 2 (19). All the girders were specified as Combination A lay-ups (Figure 1a) with the following unadjusted structural properties: $F_{bx} = 15.4$ MPa (2,240 psi), $F_{by} = 1.5$ MPa (230 psi), and $E = 11.0$ GPa (1.6 x 10$^6$ psi). The girders were braced laterally by two endwall diaphragms, midspan diaphragms, and the glulam deck, which was fastened to the girders every 0.30 m (12 in.) on center. The glulam deck panels were specified as Combination A northern red oak.

### Table 1: Recommended Allowable Design Values for Hardwood Glulam Bridge Design

<table>
<thead>
<tr>
<th>Property</th>
<th>Axis of Bending</th>
<th>Northern Red Oak</th>
<th>Red Maple</th>
<th>Yellow Poplar</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexural Strength ($F_{bx}$)</td>
<td>x</td>
<td>16.5 MPa</td>
<td>16.5 MPa</td>
<td>16.5 MPa</td>
</tr>
<tr>
<td>Stiffness ($E_s$)</td>
<td>x</td>
<td>12.4 GPa</td>
<td>12.4 GPa</td>
<td>12.4 GPa</td>
</tr>
<tr>
<td>Flexural Strength ($F_{by}$)</td>
<td>y</td>
<td>12.4 MPa</td>
<td>12.4 MPa</td>
<td>9.6 MPa</td>
</tr>
<tr>
<td>Stiffness ($E_y$)</td>
<td>y</td>
<td>11.0 GPa</td>
<td>11.7 GPa</td>
<td>9.6 GPa</td>
</tr>
<tr>
<td>Shear Strength ($F_{vx}, F_{vy}$)</td>
<td>x, y</td>
<td>1.5 MPa</td>
<td>1.4 MPa</td>
<td>1.0 MPa</td>
</tr>
<tr>
<td>Compressive Strength</td>
<td>x, y</td>
<td>6.1 MPa</td>
<td>4.2 MPa</td>
<td>2.9 MPa</td>
</tr>
</tbody>
</table>

1. Divide entries by 6.89 x 10$^{-3}$ to convert MPa to psi; divide entries by 6.89 x 10$^{-6}$ to convert GPa to psi.
2. All values are for dry-use conditions.
3. See Figure 1 for definition of x- and y-axes.
4. Northern red oak value is for Combination A lay-up; red maple and yellow poplar value for laminations lay-up similar to those described in Figure 1b or verified by ASTM 3737 procedures to have $F_{bx} = 16.5$ MPa (2400 psi) and $E_s = 12.4$ GPa (1.8 x 10$^6$ psi).
5. $F_{by}$ values are for single grade lamination lay-ups of VSR No. 1 and No. 2 lumber of each species and for nominal deck panel thicknesses of 100 mm (4 in.) and 150 mm (6 in.). Values estimated by multiplying $F_{by}$ of single laminations by 1.32 and 1.58 for northern red oak glulam beams conducted at Penn State (4).
6. $E_y$ for northern red oak is mean value of the conservative value in the NDS for single members on edge and the value from northern red oak glulam beam tests conducted at Penn State; red maple values from in-grade sampling of red maple from two locations in Pennsylvania from PennDOT Project No. SS-047 (14); yellow poplar value is average of published values for combination A lay-ups and for single members loaded on edge, NDS (9).
7. Shear strengths for each species are 1.944 times published $F_v$-value, NDS (9) for individual pieces of dimension lumber of the predominant lamination grade.
8. All values are the bearing strength of individual pieces of dimension lumber of the grade found in the face lamination, NDS (9).
with $F_c = 15.4$ MPa (2,240 psi), $F_s = 1.5$ MPa (230 psi), and $E = 11.0$ GPa (1.6 x 10^6 psi).

**Bridge Design**

The bridge superstructure has nine 203- by 743-mm (8- by 29 1/4-in.) girders spaced 965 mm (38 in.) on center (Figure 2). All girders were fabricated with 38-mm (1.5-in.) laminations. The deck, which is 152 mm (6 in.) thick, consists of panels 914 mm (36 in.) and 1220 mm (48 in.) wide by 8.54 m (28 ft) long. All panels are spaced approximately 13 mm (1/2 in.) apart to accommodate anticipated in-service moisture expansion, because the panels were fabricated at 12 ± 2 percent moisture content and are expected to equilibrate over the stream at approximately 19 percent moisture content. The 152-mm (6-in.) deck was designed as a noninterconnected deck (18,20). However, one-half of the bridge was constructed with dowels 32 mm (1 1/4 in.) in diameter to observe performance differences, if any, between the asphalt paving over the interconnected panels and the noninterconnected panels. The endwall diaphragms were 152 mm (6 in.) wide by 743 mm (29 1/4 in.) deep and extended the full 12.08-m (39.6-ft) skew length. Midspan diaphragms, 150 by 743 mm (3 by 29 1/4 in.), were installed perpendicular to the span between each pair of girders for lateral stability.

The girders were attached to the abutment with 19-mm (%-in.) anchor bolts (all bridge hardware was galvanized). The bearing design allowed vertical adjustment for proper leveling of the top surfaces of the nine beams. The deck panels were fastened to the girders with 19- by 229-mm (%-x 9-in.) galvanized lag bolts. The heads were recessed into the deck. The diaphragms were connected to the girders with three 19- by 229-mm (%-x 9-in.) galvanized lag bolts at each girder.

Oakum was installed between deck panels to prevent asphalt paving from filling the space. Before paving, a waterproof geotextile membrane was installed over the deck.

The railings and parapets design consists of 254- by 305-mm (10- x 12-in.) glulam posts spaced 1.83 m (6 ft.) on center, two 152- by 203-mm (6- x 8-in.) glulam rails, and 254- x 305-mm (10- x 12-in.) glulam curbs. The rail system is fastened with galvanized bolts and drift pins and is similar in design to that tested by Ritter et al. (21).

**Fabrication and Construction**

Most hardwood lumber is not structurally graded or available in standard sizes. This is a major challenge to the use of hardwoods in glulam applications. However, traditional hardwood manufacturers have shown interest in producing structurally graded dimension lumber.

The Penn State team procured the northern red oak logs, arranged for primary processing and drying of the lumber, and then sorted and graded the lumber in accordance with AITC’s hardwood laminating specifications (10). The team also supervised and oversaw the fabrication of the girders, deck panels, and railing materials at Unadilla Laminated Products, Inc., in Sidney, New York. As part of the fabrication process, Unadilla planed and cut all the glulam members to the required finished dimensions to minimize field

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**FIGURE 2** Superstructure layout for the 10.69-m (35-ft ½-in.) clear span northern red oak glulam demonstration bridge.
cutting after preservative treatment. The following holes were predrilled at Unadilla before preservative treatment: the deck holes for the deck-to-girder fastening; the girder holes for the girder-to-midspan diaphragms; the backwall holes for the backwall-to-girder connections, and the holes in the railings, posts, and curbs. All other holes were field drilled and field treated with creosote. Field-drilled holes were swabbed thoroughly with creosote and subsequently filled with asphaltic roof cement before inserting the fastener. All bridge connection hardware was fabricated and supplied by Unadilla, Inc. The team then shipped the laminated materials to Koppers, Inc., Muncy, Pennsylvania, where they were treated with creosote to a retention level of approximately 192.2 kg/m³ (12 pcf) with a minimum depth of penetration of approximately 6 mm (0.25 in.). Finally, the glulam members and hardware were transported to the bridge site.

The notice to proceed for construction was given on September 6, 1991. Demolition of the existing concrete bridge began on September 9, 1991. The bridge bearing seat caps were reconditioned by September 23. Much of the riprap and gabion work was completed by September 30. The girders were set and diaphragms attached by October 4, 1991. Railings and parapets and all paving were completed by October 28. On October 30, the bridge was load tested with two 75,000-lb triaxle trucks. On November 1, 8 weeks after the notice to proceed, the road was reopened to traffic.

Special care was necessary in the construction phase to ensure proper mating of the deck to the girder. All deck panels had to be properly mated to the girder before insertion and tightening of the lag bolts. This can be accomplished by careful sequencing of the fastener application and by preloading the deck panel with construction equipment before fastening and properly torquing the lag bolts.

Dimensional stability was also a concern in the fabrication/construction phase. Laminating procedures require lumber to be at moisture content less than 16 percent. In service treated lumber in the deck panels is expected to equilibrate at approximately 19 percent moisture content. Thus, adequate spacing, the magnitude of which is somewhat species dependent, must be specified between deck panels. Because some expansion occurs during creosote treatment, panel widths at the site should be measured before installation for necessary panel spacing adjustments in the field.

Cost

The total cost of the bridge, including all materials, development, and monitoring costs, was approximately $250,000. The design, construction, and engineering costs, which totaled $140,000, were higher than normal for a "spec" bridge. Laminating costs were $40,000. Since this bridge was a demonstration/experimental bridge, it was the first of its kind. Consequently, uncertainties with respect to design, fabrication, and construction procedures required more time and effort.

Once the standard designs and specifications are available in 1993, these costs should decline significantly, thus making timber bridges more cost competitive. In addition, these costs include evaluation of the bridge's structural responses and overall performance over a 3-year period.

Performance Test Results

The predicted live load deflection, assuming no composite behavior between the deck and girder, girder E-value of 11.0 GPa (1.6 x 10⁶ psi), and an HS25 or ML80 load, was 22 mm (0.85 in.). A load test with two 33.4-kN (75,000-lb) triaxial trucks, located to produce maximum deflection, produced an actual maximum deflection of 14 mm (0.55 in.). Lower actual versus predicted deflection is probably due to (a) neglecting composite action, (b) using an E-value that is somewhat lower than found in previous work [Shaffer et al. (11) reported E-values of 13.1 GPa (1.9 x 10⁶ psi) for northern red oak beams], and (c) using an E-value that is considerably lower than that found in the actual bridge stringers by test [the average E-value, determined by static loading, of each board used in the bridge girders was 15.5 GPa (2.2 x 10⁶ psi)].

Predicted live load deflection using an E-value of 13.1 GPa (1.9 x 10⁶ psi) and 15.5 GPa (2.2 x 10⁶ psi) equals 18 mm (0.72 in.) and 16 mm (0.62 in.), respectively. Fourteen months' data indicate that creep deflections are negligible (less than 1 mm).

Some small reflective cracks have formed in the bituminous paving directly above the interface between adjacent deck panels. There is no noticeable difference in the amount of cracking in the doweled and nondoweled ends of the deck. The cracking is most likely due to the expansion spacing 13 mm (0.5 in.) wide provided between deck panels during construction. Dimensional changes in the deck panels have been observed and measured monthly since construction was completed. After 1 year, the spacings have reduced to approximately 40 percent of the original width. This observation suggests that the spacing between deck panels may be reduced to 6 mm (0.25 in.).

HARDWOOD GLULAM BRIDGE STANDARDS

In May 1993 the Pennsylvania Department of Transportation published the BLC-560 series Standards for Hardwood Glulam Timber Bridge Design (1) for clear spans of 5.49 to 27.4 m (18 to 90 ft). The standards include designs and specifications for the design, fabrication, treatment, handling, and erection of components and assemblies for hardwood glulam bridge substructures and superstructures. The standards are flexible and include provisions for hardwood or nonhardwood substructures used in conjunction with hardwood superstructures. The standards also include selection tables for clear spans of 5.49 to 27.4 m (18 to 90 ft).

SUMMARY

The technological basis for designing hardwood glulam timber highway bridges of northern red oak, red maple, or yellow poplar has been developed. A demonstration highway bridge using northern red oak glulam members for the superstructure, including parapets and railings, has been successfully designed, fabricated, erected, and load tested. Standard designs (BLC-560 series) have been developed for northern...
red oak, red maple, and yellow poplar glulam bridges with clear spans ranging from 5.49 to 27.4 m (18 to 90 ft). The standards are available from the Pennsylvania Department of Transportation.

ACKNOWLEDGMENTS

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REFERENCES

Experimental Evaluation of Stressed Timber Bridge Systems

L. Shelton Barger, Jr., Roberto Lopez-Anido, and Hota V. S. Gangarao

The stiffness and transverse load distribution variations of stressed timber bridge systems, including the shear lag phenomenon for the Tee and Box superstructure configurations, two stringer spacings, and two prestress levels, were examined. The tests were carried out for static loads that were applied at midspan on both interior and exterior stringer locations. Deflections and strains at different transverse locations on the deck and stringers were obtained. The analysis of the data provides helpful information to evaluate stiffnesses of stressed Tee and Box timber bridges. Composite moments of inertia of the stringers were obtained from the experimental flexibility coefficients. Shear lag in flanges was evaluated by accounting for the effective flange width of an individual composite beam. The experimental strain variations were used to validate this model.

Modern stressed timber bridges have proved to be an efficient and cost-competitive alternative to conventional bridge construction. In the field, 2- to 4-in.-thick lumber laminations that are dried and treated with creosote are squeezed by high-strength steel rods to develop a stress-laminated timber deck. Interlaminar frictional forces are induced on the lumber lamination faces by the compressive or prestressing (squeezing) forces. These frictional forces are responsible for preventing vertical slippage of the lumber laminations (1).

A properly designed and fabricated stress-laminated timber deck is economical and safe for bridges with spans of less than 9 m (30 ft) (2). Stressed timber bridges of greater span lengths are feasible by adding stringers or beams to a stressed deck. A variety of manufactured timber beams such as laminated veneer lumber, glulam, and parallam can be used for stringers. Therefore stressed timber systems for spans longer than 9 m (30 ft) are possible by modifying the basic stressed deck component with the inclusion of timber stringers to the stressed deck at certain intervals (Tee system) or stringers and a bottom deck (Box system), as shown in Figure 1.

Although the stress-lamination method of building timber bridges has some significant advantages over traditional timber bridge construction, the use of timber for bridges has some limitations that we should address. A stress-laminated system is structurally orthotropic. In the transverse direction, the large number of laminations across the width reduces the transverse stiffness to a small fraction of the longitudinal stiffness. The objective of this study is to understand the stiffness and transverse load distribution variations of the stressed timber systems, including the shear lag phenomenon for the Tee and Box superstructure configurations, for two stringer spacings, and for two prestress levels. The tests described in this paper consisted of static loads applied at midspan on both interior and exterior stringer locations. Deflections and strains at different transverse locations on the deck and stringers were obtained. The analysis of the data from 16 tests provides helpful information for the evaluation of the actual stiffness of Tee and Box stressed timber bridges. Linear regression analyses were conducted on the experimental deflections to compute flexibility coefficients. Composite moments of inertia of the stringers were obtained from the flexibility coefficients. The shear lag phenomenon is accounted for, in a design-oriented approach, by computing the effective flange width of an isolated composite beam. The experimental strain variations were used to validate this model.

TEST METHODOLOGY AND CONFIGURATIONS

The static testing of scale model Tee and Box stressed systems without creosote treatment was conducted in the Major Units Laboratory at West Virginia University. All the systems were made of northern red oak decks (Grade 3) of nominal 5- x 15-cm (2- x 6-in.) laminations and southern pine glulam beams (24F-V3, SP/SP) 6.10 m (20 ft) long and 62.5 x 12.7 cm (24.625 x 5 in.). The moduli of elasticity (MOE) for the bridge components were obtained from the linear portion of the load-deflection curve following the Standard Methods of Static Tests of Timbers in Structural Sizes (ASTM D198-84). The mean MOE obtained for the northern red oak boards was $E_D = 11,170$ MPa (1,620,000 psi), with a coefficient of variation of 15.3 percent. The mean MOE obtained for the southern pine glulam beams was $E_s = 9,060$ MPa (1,314,000 psi).

Tee and Box systems with a span $L = 5.89$ m (19 ft 4 in.) were built with two different stringer configurations. The three-stringer models had a center-to-center stringer spacing $S = 135$ cm (53 in.), and the four-stringer models had $S = 81$ cm (32 in.). A predetermined butt joint pattern of one butt joint every four laminations was used to assemble the decks. The actual deck thickness was 14 cm (5.5 in.). A good approximation for the deck longitudinal modulus was the mean MOE obtained for the boards ($E_D$) (3). Each model was tested with two prestress levels: 0.690 MPa (100 psi) and 0.345 MPa (50 psi).

Midspan deflections and strains were obtained at different transverse locations on the decks and on the stringers. The deflections and strains were measured with LVDTs and strain gauges, respectively, at load increments of 4.45 kN (1,000 lb).
The loading was applied with a hydraulic ram and measured by a calibrated load cell. The applied load cases for each configuration included a concentrated load on an interior stringer and a concentrated load on an exterior stringer. The concentrated load was applied over an area of $20 \times 14 \text{ cm} (8 \times 5.5 \text{ in})$. The transverse prestress level of the decks was monitored during the tests.

**ANALYSIS OF EXPERIMENTAL DEFLECTIONS AND EVALUATION OF TRANSVERSE LOAD DISTRIBUTION FACTORS**

Load-deflection ($P$-$\delta$) curves for the models at the lower stress level under a load applied on an interior stringer are shown in Figure 2. The reductions in flexibility from a Tee to a Box system and from a three- to a four-stringer configuration can be seen in Figure 2. Flexibility coefficients ($\delta_i/P$) were obtained from linear regression analyses on the experimental deflections and are presented in Tables 1 and 2 for the three- and four-stringer models, respectively. The results from Tables 1 and 2 allow us to evaluate the flexibility response of the models at different transverse locations and under different loading conditions. Maximum flexibility coefficients for the Box models are approximately 87 percent of the corresponding values for the Tee models at an interior stringer and 78 percent at an exterior stringer.

**ANALYSIS OF EXPERIMENTAL STRAINS AND INVESTIGATION OF THE SHEAR LAG EFFECT**

Load-strain curves for the three-stringer models under a concentrated load applied on the interior stringer are shown in Figure 3. After an initial slack, a linear relation is observed in the range of 8.90 kN (2,000 lb) to 26.69 kN (6,000 lb). The transverse variations of experimental strains under a load $P = 17.80$ kN (4,000 lb) applied on the interior stringer are plotted in Figure 4 for the three-stringer models and in Figure 5 for the four-stringer models. In the upper deck, a nonuniform compressive strain distribution is observed, and it is more pronounced for the Tee than for the Box system. A severe local effect in the variation of the tensile strains is noted in
TABLE 1 Flexibility Coefficients for the Three-Stringer Models

<table>
<thead>
<tr>
<th>Load level</th>
<th>Force Tee Box</th>
<th>Force Tee Box</th>
<th>Force Tee Box</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.690</td>
<td>0.1144</td>
<td>0.0059</td>
<td>0.0873</td>
</tr>
<tr>
<td>0.345</td>
<td>0.1242</td>
<td>0.0066</td>
<td>0.1048</td>
</tr>
<tr>
<td>#2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.690</td>
<td>0.0076</td>
<td>0.0842</td>
<td>0.0084</td>
</tr>
<tr>
<td>0.345</td>
<td>0.0055</td>
<td>0.0937</td>
<td>0.0070</td>
</tr>
</tbody>
</table>

Obtained from linear regression on the experimental curves of load versus deflection. Concentrated load applied on exterior stringer #1. Concentrated load applied on interior stringer #2.

1 MPa = 145.04 psi
1 mm = 0.0394 in
1 KN = 224.81 lbs

TABLE 2 Flexibility Coefficients for the Four-Stringer Models

<table>
<thead>
<tr>
<th>Load level</th>
<th>Force Tee Box</th>
<th>Force Tee Box</th>
<th>Force Tee Box</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.690</td>
<td>0.1020</td>
<td>0.0184</td>
<td>0.0050</td>
</tr>
<tr>
<td>0.345</td>
<td>0.1218</td>
<td>0.0114</td>
<td>0.0023</td>
</tr>
<tr>
<td>#2</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>0.690</td>
<td>0.0185</td>
<td>0.0820</td>
<td>0.0150</td>
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<tr>
<td>0.345</td>
<td>0.0114</td>
<td>0.0902</td>
<td>0.0143</td>
</tr>
</tbody>
</table>

Obtained from linear regression on the experimental curves of load versus deflection. Concentrated load applied on exterior stringer #1. Concentrated load applied on interior stringer #2.

1 MPa = 145.04 psi
1 mm = 0.0394 in
1 KN = 224.81 lbs

TABLE 3 Transverse Load Distribution Factors

<table>
<thead>
<tr>
<th>System</th>
<th>Prestress Three-stringer models</th>
<th>Four-stringer models</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Level (MPa)</td>
<td>ω₁</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ω₁</td>
</tr>
<tr>
<td>Tee</td>
<td>0.690</td>
<td>0.07</td>
</tr>
<tr>
<td></td>
<td>0.345</td>
<td>0.05</td>
</tr>
<tr>
<td>Box</td>
<td>0.690</td>
<td>0.09</td>
</tr>
<tr>
<td></td>
<td>0.345</td>
<td>0.07</td>
</tr>
</tbody>
</table>

Computed from Equation 1.
Applicable to both exterior stringers due to symmetry.
Evaluated at the location of the concentrated load.
1 MPa = 145.04 psi

the lower deck of the Box models under the applied load. The influence of the prestressing level in the transverse strain distribution can be observed in Figures 4 and 5. Because no slip between the laminations was observed, the steep strain drop is attributed to the shear lag phenomenon that is more critical in stressed timber bridges than in conventional decks. This inability of the stressed deck to transmit shear stresses reduces the "effective width" in a stressed timber deck design.

EVALUATION OF COMPOSITE BEAM STIFFNESS AND COMPUTATION OF THE EFFECTIVE FLANGE WIDTH

In the design of a Tee or a Box stressed timber bridge, it is necessary to evaluate the composite moment of inertia of an individual beam formed by the stringer and the effective portion of the deck. Composite beam moments of inertia for the
FIGURE 3 Load-strain curves for the three-stringer models under a load applied on the interior stringer.

FIGURE 4 Transverse strain distribution for the three-stringer models under a load applied on the interior stringer.

FIGURE 5 Transverse strain distribution for the four-stringer models at the lower stress level under a load applied on an interior stringer.

different configurations were obtained by equating the experimental flexibility coefficients from Tables 1 and 2 with the corresponding normalized beam deflection formula modified by the appropriate load distribution factor from Table 3. Then for the ith stringer we get

\[ I_c = \frac{L^3}{48E_s} \left( \frac{\delta_i}{P_{\text{EXP}}} \right) \]

The resulting composite moments of inertia \( I_c \) are presented in Table 4 relative to the stringer inertia \( I_s \). The Box systems develop an increment in \( I_c \) with respect to the corresponding Tee systems by about 4 percent for interior stringers and 8 percent for exterior stringers.

Regarding the design of a timber bridge system as an isolated composite beam, the reduction in the collaborating deck width due to the shear lag effect was evaluated. Total effective flange widths \( (b_E) \) were obtained from the composite moment of inertia by assuming an equivalent homogeneous T-

| TABLE 4 Composite Moment of Inertia* and Effective Flange Width** |
|-----------------|-----------------|-----------------|
| System        | Prestress level (MPa) | Three-stringer models | Four-stringer models |
|                | \( I_c / I_s \) | \( b_E / S \) | \( I_c / I_s \) | \( b_E / S \) | \( I_c / I_s \) | \( b_E / S \) |
| Tee           | 0.690           | 1.85            | 0.37            | 1.53            | 0.38            | 1.41            | 0.31            |
|               | 0.345           | 1.74            | 0.32            | 1.51            | 0.37            | 1.33            | 0.28            |
| Box           | 0.690           | 1.96            | 0.32            | 1.59            | 0.25            | 1.52            | 0.24            |
|               | 0.345           | 1.78            | 0.17            | 1.54            | 0.24            | 1.44            | 0.22            |

*Computed from Equation 2, and expressed relative to the stringer inertia.

**Obtained from the composite moment of inertia by assuming an equivalent homogeneous section, and expressed relative to the stringer spacing.

Spacement center to center of stringers: \( S = 135 \) cm.

Spacement center to center of stringers: \( S = 81 \) cm.

1 MPa = 145.04 psi

1 cm = 0.394 in.
section for the Tee systems and an equivalent homogeneous I-section for the Box systems. The resulting effective flange widths are given in Table 4 relative to the stringer spacing. The reduction in effective width of an exterior stringer relative to an interior stringer is more pronounced for the Tee system (22 percent) than for the Box system (6 percent). The variation of effective deck width with the level of prestressing is about 9 percent for the Tee models and 6 percent for the Box models. The four-stringer Box system presents an effective width that is around 65 and 78 percent of the corresponding values of the Tee system for an interior and an exterior stringer, respectively. The reduction in effective flange width for the Box configurations is justified because of the more pronounced shear lag developed in the bottom deck in the proximity of the load location, as shown in Figures 4 and 5. For the four-stringer Tee system the effective width can be expressed as $\frac{1}{20}$ of the span or 2.2 times the deck thickness, whereas for the Box system the effective width can be stated as $\frac{1}{50}$ of the span or 1.4 times the deck thickness. The effective flange widths computed in Table 4 account for the influence of the butt joint pattern in the deck.

CONCLUSIONS

From our experimental data, and as preliminary design guidelines for a Tee system, the effective flange width of S/3 for the interior stringers and S/4 for the exterior stringers are recommended. Similarly, for the Box system, effective flange widths of S/4 for the interior stringers and S/5 for the exterior stringers are recommended. In the field, over a long period of time, we expect the prestress level to stabilize near 0.345 MPa (50 psi). For this reason, we recommend this prestress level for design purposes. In the field the diaphragms effectively contribute to increase the transverse stiffness of the Tee system. The curbs also provide a substantial stiffening of the exterior stringers. These effects, which yield a better transverse load distribution leading to higher load sharing by the exterior stringers, have been excluded from the results reported in this paper and will be presented as a sequel to this study. Effects due to loads applied between stringers will also be discussed in a separate paper.

The analysis of the data provides helpful information for the evaluation of the stiffness of stressed Tee and Box timber bridges, including the transverse load distribution factors and the effective flange widths. The analysis of the transverse strain variation provides a valuable insight toward the understanding of the reported results. Mathematical models representing the above behavior of Tee and Box systems are being developed and will be presented as an extension of this study.

ACKNOWLEDGMENT

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REFERENCES


The contents of this paper reflect the views of the authors and do not reflect the official views of the sponsor.
Issues Related to Administration of Low-Volume Roads in Developing Countries

JACOB GREENSTEIN

The predominant need in the administration of rural roads is to improve the maintenance and performance of the existing network rather than to construct new roads. To achieve this goal, an economic analysis of costs and benefits related to the level of accessibility and the economic life of the road network is normally required. In rural areas where the road is a component of an area development program and traffic volumes are low (less than 50 vehicles per day), a socioeconomic methodology that examines the relationship between road accessibility, agricultural and forestry production, and social services has been applied to evaluate costs and benefits of investments of the whole program, including its road improvement component. Rural investment is most efficient when the most economic type of roadway and the complementary agricultural/forestry social investments are jointly optimized. The principal benefits achieved are reduced transport costs, increased area of agricultural land in production, increased yield per unit area, and all-weather accessibility. One of the most important tasks to be done by local governments is the condition inventory and evaluation of the road network. Other issues discussed include design of low-cost bridges and water crossings, optimization of routine and periodic maintenance expenditures, and the application of environmental procedures in the administration of low-volume roads.

Rural roads are often significant in terms of mileage in the overall network, tonnage transported, and socioeconomic value. Two-thirds to three-quarters of the world's roads are low-volume roads (LVRs). LVRs are usually constructed and administered in an environment of minimum investment and are usually the first and primary link between raw materials and the world market. One of the Inter-American Development Bank (IDB) objectives is to assist rural roads authorities to develop the most practical and economical road improvement investment programs and to administer efficiently these rural road networks (1-4), including their environmental impacts.

The socioeconomic evaluation criteria used to assess road improvement distinguish between existing roads with considerable traffic and roads with a low volume of traffic. Both procedures are presented in the paper in the context of the following issues related to road planning and administration: alternatives for rural road improvement, inventory and road evaluation aided by means of knowledge base expert systems, simplified socioeconomic procedures for road improvement, environmental issues related to rural roads improvement and maintenance, planning and administration of forestry roads, design of low-cost bridges and water crossings, and optimization of routine and periodic pavement maintenance expenditures of gravel roads.

ALTERNATIVES FOR ROAD IMPROVEMENT

The main objective of rural road planning is to develop an investment program that can efficiently allocate and use available resources by developing priorities for road improvement and maintenance activities (1-4). To achieve this, a socioeconomic analysis of costs and benefits related to the level of accessibility and the economic life of the road network is carried out (5). The analysis requires the estimation of economic costs and benefits resulting from the improvements of rural road networks. These normally consist of eight different types of pavements:

1. Asphaltic concrete or sand asphalt,
2. Surface treatment (single, double, and sometimes triple),
3. Chemically stabilized base course (lime, cement) with or without blacktop,
4. Crushed stone or gravel base course with or without blacktop,
5. Gravel or subbase without blacktop,
6. Stone roads,
7. Compacted selected (lateritic) local materials, and
8. Earth roads.

The first seven types of roads are designed to provide all-weather accessibility.

Asphalt roads usually have representative design speeds of 50 to 90 km/hr, whereas gravel roads are usually in the range 25 to 50 km/hr. These upper and lower limits represent design speeds for level and mountainous terrains, respectively, and may also vary depending on the local topographical and environmental conditions. The total width of an asphalt road is usually between 7.2 and 9.7 m. The design speed of a Type 8 earth road is usually between 20 and 30 km/hr, and its width varies between 3 and 4 m.

For each uniform road link there are several alternative improvements. For example, there are more than seven alternatives to upgrade Road Type 8 because there are seven types of higher class road, 1 to 7, and each of these may be varied to provide additional alternatives. To determine the most economic upgrade of the rural road network, it is necessary to analyze a wide range of road improvement alternatives and to select the combination of network betterment that results in the highest marginal rate of return. In other words, for each improvement alternative of the road network, it is necessary to determine, for the life of the projects, the streams of costs and benefits and to calculate the economic priority indicators such as internal rate of return (IRR), first year rate of return (FYRR), net present value (NPV), and

benefit cost (B/C) ratio. The economic indicators are calculated for each road link and for the improvement of the entire network.

INVENTORY AND ROAD EVALUATION

Good road management requires continual updating of information about the road network. This is achieved by means of a road inventory that identifies each road link and evaluates both the engineering characteristics and the condition of the road elements, including alignment, drainage, pavement materials, and surface condition. During the inventory, notes are kept indicating the need for such road improvements as pavement strengthening, replacement of drainage facilities, shoulder improvements, and other emergency work not covered by routine or periodic maintenance. The inventory team normally analyzes and evaluates about 40 road parameters (see Table 1). Each parameter has on the average three to five quantitative or qualitative values that should be evaluated in the field. For example, the surface and drainage conditions are evaluated qualitatively on a scale of 1 to 5, with 1 being the worst and 5 the best (5). Also, if some of the road elements are in poor or bad condition and accessibility is limited, the inventory team is asked to determine what kind of road improvement is required (e.g., pavement overlay, pavement strengthening with base course with or without asphalt, raising of the surface elevation, or replacement or addition of culverts).

The inventory team has to make accurate assessments of at least 150 road engineering indicators, and an inaccurate evaluation of any of these could distort the planning of improvement, maintenance, and rehabilitation. The author’s experience of more than 15 years in planning and designing rural roads in developing countries indicates that despite the efforts made in the training of evaluation teams, a strong need exists to screen, verify, and adjust the information gathered in the field during the inventory. To assist the process, and to obtain greater uniformity and reliability in the results, it is helpful to use a logic analysis expert system computer program.

Once the road links are identified and evaluated, the road network’s requirements for improvement and maintenance are determined (5). Some partial instructions and an example of a road inventory carried out in Ecuador are presented in Table 1. Special expertise is needed to conduct field evaluations of the engineering characteristics of the subgrade and pavement systems given in Table 1 (Section 4, Boxes 401–422). In this section, the inventory team has to determine among other things the subgrade and pavement California bearing ratio (CBR), the type and severity of surface distress, and the quality or efficiency of the routine maintenance work. The three most common types of surface distress and their severity are identified in Boxes 407–413. The following are types of surface distress for roads with a blacktop (the eight types of surface distress most commonly found in rural roads without blacktop are italic):

1. Potholes,
2. Weathering/raveling,
3. Alligator cracking,
4. Slippage cracking,
5. Block cracking,
6. Edge failure,
7. Longitudinal/transverse cracking,
8. Lane/shoulder drop,
9. Bumps and sags,
10. Depression,
11. Corrugation,
12. Shoving,
13. Swelling,
14. Rutting,
15. Bleeding, and

While in the field, the engineering evaluation team is also required to identify the most practical and economical alternative for road improvement (Table 1, Boxes 501–515). Targeted improvements include the rehabilitation of new bridges and culverts (Boxes 501–507), determination of the minimum embankment rise in elevation to prevent flooding or overtopping during the rainy season (Box 508), or improvement of the geometry or drainage characteristics of the roads by means of excavation (Box 509). Another conclusion is related to urgent needs for pavement repair or maintenance (Boxes 510–515). The inventory team must determine the three most urgent needs for pavement repair. The 11 most common types of pavement repairs on rural roads are pothole filling; pavement strengthening with an additional 7 to 30 cm of granular material with or without blacktop; shoulder improvement; grading, shaping, and compacting granular pavement without blacktop; crack sealing; skin or partial depth patching; full-depth patching; application of heat and sand; surface treatment; asphalt overlay; and pavement reconstruction.

Expert systems are interactive computer programs that use expert knowledge to obtain enhanced levels of performance in a narrow problem area (6). They contain a collection of specific domain knowledge called a knowledge base (KB) and a general problem-solving ability called an inference engine. Expert system building tools are now available for microcomputers that allow experts in other fields to prepare expert systems in their areas of specialization. These building tools, known as shells, are rapidly emerging as a new class of computer program, joining the ranks of word processors, data base managers, and spreadsheets.

The need for a rural road inventory expert system arose because engineers who have been taught the basic mechanics of the procedures have occasionally performed poorly, failing to notice gross inconsistencies in both the data and the conclusions. The rural road inventory expert system package seeks to upgrade the performance of engineers with limited experience. It was prepared using commercially available shells and contains KBS for analyzing the rural inventory data as well as its conclusions and recommendations (5).

SIMPLIFIED SOCIOECONOMIC PROCEDURE FOR ROAD IMPROVEMENT

Saving of transportation costs is not always sufficient to justify rural road improvement in countries such as Ecuador, Peru,
and Bolivia (3,4). The rural economic growth of these countries has often been substantially hindered by poor road accessibility, which has resulted in increased vehicle operating costs and traffic hazards. A socioeconomic methodology has been developed on the basis of the relationship between road accessibility, agricultural production, and economic and social indicators for rural improvement. This can be used to evaluate the benefit of investments from road improvement in rural areas (3,4). The principal conclusion is that rural investment can only be optimized when the most economic type of road network and the complementary agricultural investments are determined simultaneously. The main benefits are

- Reduced transport costs through the substitution of small and uneconomical vehicles, animal transport, or river boats by larger and more economical motor vehicles;
- More effective use of agricultural land by conversion from subsistence farming to commercial production;

<table>
<thead>
<tr>
<th>TABLE 1 Sample of Rural Road Inventory Data Sheets (7)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ENGINEERING CHARACTERISTICS</td>
</tr>
<tr>
<td>NO. OF MONTHS WITH NO ACCESSIBILITY</td>
</tr>
<tr>
<td>NO. OF EXISTING BRIDGES</td>
</tr>
<tr>
<td>NO. OF EXISTING CULVERTS</td>
</tr>
<tr>
<td>TYPE OF TERRAIN (1-level, 2-hilly, 3-mountainous)</td>
</tr>
<tr>
<td>TYPE OF ROAD SURFACE (1-asphalt concrete, 2-asphalt surface treatment, 3-chemically stabilized, 4-crushed stone base, 5-gravel road, 6-stone road [empedrado], 7-compact selected local materials, 8-earth road)</td>
</tr>
<tr>
<td>EFFECTIVE, OR USED ROAD WIDTH (MET)</td>
</tr>
<tr>
<td>TOTAL ROAD WIDTH INCLUDING SHOULDERS (MET)</td>
</tr>
<tr>
<td>SHOULDER WIDTH (MET)</td>
</tr>
<tr>
<td>TYPE OF SHOULDER (0-does not exist, 1-paved, 2-granular or stabilized, 3-soil, 4-others)</td>
</tr>
<tr>
<td>ROAD CLASSIFICATION (1-main, 2-principal rural, 3-secondary rural, 4-penetration)</td>
</tr>
<tr>
<td>FIELD EVALUATION OF THE ROAD ELEMENTS</td>
</tr>
<tr>
<td>ACTUAL TRAVELING SPEED (km/h)</td>
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<tr>
<td>AVERAGE DAILY TRAFFIC</td>
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<tr>
<td>CONDITION OF THE HORIZONTAL ALIGNMENT (1-very bad, 2-bad, 3-poor 4-fair, 5-good)</td>
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<tr>
<td>POSSIBILITY OF IMPROVING HORIZ. ALIGNM. (1-3)</td>
</tr>
<tr>
<td>LONGITUDINAL GRADE (1-over 101 to 5-less than 2%)</td>
</tr>
<tr>
<td>SURFACE CONDITIONS (1-very bad to 5-good)</td>
</tr>
<tr>
<td>CONDITION OF SHOULDERS (1-very bad to 5-good)</td>
</tr>
<tr>
<td>VISIBILITY OF TAKEOVER (1-very bad to 5-good)</td>
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<tr>
<td>DRAINAGE CONDITIONS (1-very bad to 5-very good)</td>
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<tr>
<td>SUBGRADE CONDITIONS (1-very bad to 5-good)</td>
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<tr>
<td>ALTITUDE ABOVE SEA LEVEL</td>
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<tr>
<td>CLIMATE CONDITIONS (1-tropical, 2-subtropical, 3-arid)</td>
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<tr>
<td>EVALUATION OF THE SUBGRADE AND PAVEMENT SYSTEM SOIL CLASSIFICATION (26 possibilities)</td>
</tr>
<tr>
<td>ESTIMATED SUBGRADE CBR</td>
</tr>
<tr>
<td>ESTIMATED PAVEMENT CBR (unsurfaced roads)</td>
</tr>
<tr>
<td>DETERMINATION OF THE THREE MOST COMMON SURFACE DISTRESSES (1-16 and 1-8 for roads with and without blacktop)</td>
</tr>
<tr>
<td>THE SEVERITY OF EACH DISTRESS (1-high, 2-medium, 3-low)</td>
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<tr>
<td>EFFICIENCY/QUALITY OF PAVEMENT MAINTENANCE (1-poor, 2-limited, 3-adequate)</td>
</tr>
<tr>
<td>ESTIMATED DATE OF LAST PAVEMENT OVERLAY OR CONSTRUCTION (month/year)</td>
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<tr>
<td>HAULAGE DISTANCE OF MATERIALS (km)</td>
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<tr>
<td>IDENTIFICATION OF ROAD IMPROVEMENT AND MAINTENANCE Needs NUMBER OF NEW BRIDGES NEEDED</td>
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<td>TOTAL LENGTH OF NEW BRIDGES (meters)</td>
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<tr>
<td>ESTIMATED NUMBER OF NEW CULVERTS</td>
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<td>STRENGTHENING, RAISING OF EMBANKMENT HEIGHT (1-over 1.0 meters to 5-0.0 meters)</td>
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<tr>
<td>AVERAGE EXCAVATION OR CUT DEPTH NEEDED TO IMPROVE THE ROAD CHARACTERISTICS (1-over 8.0 meters to 5-less than 0.6 meters)</td>
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<tr>
<td>URGENT NEED OF PAVEMENT REPAIR OR SPECIAL MAINTENANCE NEEDS (1 to 10)</td>
</tr>
<tr>
<td>URGENT NEED OF PAVEMENT REPAIR OR SPECIAL MAINTENANCE NEEDS (1 to 10)</td>
</tr>
<tr>
<td>URGENT NEED OF PAVEMENT REPAIR OR SPECIAL MAINTENANCE NEEDS (1 to 10)</td>
</tr>
</tbody>
</table>

* Suggested by the computer: 02 denotes the number of activity of pavement repair
* Suggested by the computer: 17 denotes the thickness of the new base in cm.
• Increased yields through the introduction of more modern farming equipment, fertilizers, pesticides, and technical assistance;
• Substitution of high-value perishable crops grown for the domestic market for long-life staple crops; and
• All-weather accessibility, which permits lower storage requirements and related inventory costs as well as the harvesting of crops when they are ready for marketing, regardless of weather conditions.

When these additional factors generate more benefits than the total expenditures required during the lifetime of the road and produce an IRR and FYRR greater than 12 percent, the road investment is normally considered justified (1,3,4).

DETERMINATION OF THE MOST ECONOMICAL TYPE OF ROAD

Determination of the optimum type of road for each level of traffic is done by analyzing the relationship between the total transportation cost and the traffic volume. The total transportation cost for a given road network includes reconstruction, rehabilitation, maintenance, and vehicle operating cost (VOC) expenditures during the economic lifetime of the project. During this period, most of the benefits of the complementary agricultural investment can be developed, justifying the road network's improvement (1,3,4). For any given traffic projection, the total transportation cost varies with type of road, surface conditions, rehabilitation and maintenance costs, the engineering properties of the existing soils and materials, and the local environmental characteristics. The conclusions of this road screening or threshold analysis (2,3,7) are summarized in the following table:

<table>
<thead>
<tr>
<th>Road Type (Minimum Transportation Cost)</th>
<th>Traffic Volume (Vehicles per Day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8 (earth)</td>
<td>less than 50</td>
</tr>
<tr>
<td>5 or 6 (gravel or stone)</td>
<td>50–100</td>
</tr>
<tr>
<td>3 or 4 (crushed or stabilized gravel, stone)</td>
<td>101–250</td>
</tr>
<tr>
<td>2 (asphalt surface treatment)</td>
<td>over 250</td>
</tr>
</tbody>
</table>

According to these tabulations, when the traffic volume is between 50 and 250 vehicles per day, it is better to construct an all-weather gravel road than an asphalt treatment road. When the ADT is less than 50 vehicles per day, savings in VOC and road maintenance costs are not usually sufficient to justify the improvement of a dry season dirt road to an all-weather gravel road. Improving traveling safety and reducing road accidents should also be considered in road investment programs, since traveling safety is related mainly to roads with divided lanes and controlled accesses (1). It has not yet been proven that improvement of low-volume roads results in significant economic savings due to accident reduction, and therefore this is not considered in this paper.

COMPLEMENTARY RURAL INVESTMENT COSTS

Segments of the road network that play a principal role in connecting production centers with markets and that carry sufficient projected traffic volume to justify an upgrade are analyzed in terms of the net increase in the economic value of the production of goods and services that result from the road investment. In this case, an improvement of the road accessibility, such as an upgrading from a dry-season dirt road to an all-weather road, may contribute to increasing the value of production in the zone affected by road improvement through lower costs of inputs, lower costs of marketing, and higher farm gate yield and prices.

In rich agricultural areas, road improvement can often be economically justified without the need for complementary rural investments (3,4). In this case, the road improvement increases the value of production, lowers storage requirements and related inventory costs, and allows harvesting the crops when they are ready for marketing, regardless of weather conditions. On the other hand, in many rural areas of countries such as Ecuador, Peru, and Bolivia, complementary agricultural investment is essential to justify such road projects. In other words, in many rural development projects, road rehabilitation and agricultural development (a complementary investment) are necessary to economically justify rural road investments.

AGRICULTURAL AND TRANSPORTATION BENEFITS OR VALUE ADDED ACHIEVED THROUGH ROAD IMPROVEMENT

Value Added Resulting from Improvement in the Agricultural Farming System

Three major farming systems were identified: traditional, semitechnical, and technical. The traditional system is characterized mainly by the use of the family work force. Seeds are from the last harvest, and neither fertilizers nor technical assistance is used. Yields are low, and a large portion of the harvest is for local subsistence. Approximately 72 percent of the cultivated areas in the seven provinces studied in Ecuador in 1985 were identified as traditional (3,4). The semitechnical system is characterized by the use of machinery for land preparation, fertilizers in a selective form, and improved seed. The farmer uses limited technical assistance and credit. Yields are varied, and the harvest is frequently mechanized. Approximately 27 percent of the area studied in 1985 was identified as semitechnical or partially mechanized. The technical system is totally mechanized, capital-intensive, and characterized by total control of seed quality and the use of fertilizers and chemical elements. The farmer makes extensive use of technical assistance and credit; yields are very high, and the harvest is frequently mechanized. An estimated 1 percent of the studied area was identified as technical in 1985.

The main constraints to the improvement of the agricultural system from traditional to semitechnical are the lack of adequate infrastructure (principally all-weather roads for market access), the use of inputs (such as improved seed and fertilizers, which should be brought in from outside the zone), and the introduction of technical assistance. To estimate the value added by changing the farming system, a production function was developed for about 60 main agricultural products in the area studied (e.g., coffee, cacao, bananas, citrus fruit, potatoes, garlic, onions, and tomatoes). For each agricultural item, the production function related the production cost and yields to the farming system. A high rate of return can be achieved by improving the agricultural method in conjunction with improvements to the road network. This economic return on the road component is approximately 30 to 35 percent, or
U.S. $18,000/km/year, compared with the average investment in gravel road improvement. This high annual benefit can usually be achieved during an approximately 7- to 10-year transition period in which the traditional agricultural production system is improved into the semitechnical method.

Value Added Resulting from Reducing Transportation Costs

Savings on transportation costs in rural South America are usually obtained in the following ways: (a) by reducing the VOCs by using roads with better surface conditions or more economical vehicles (3,4,8) and (b) by reducing the cost of transporting agricultural products by using motorized vehicles on new roads instead of animals in areas where roads do not now exist (3,4).

The transportation of agricultural products from rural areas to market in South America is often done by pickup or light trucks that carry up to 1800 kg. The transportation cost in Ecuador, in normal road conditions, in 1985 was U.S. $0.17/ton/km (4). This vehicle transportation unit cost was only one-fifth to one-sixth of the cost of animal transport by mules. Therefore, the annual VOC saving of an average ADT of 40 vehicles is approximately U.S. $3,500/year/km, or approximately 6 to 7 percent of the investment needed to construct a gravel road. In rural tropical areas in Ecuador, the Selva of Peru, and the eastern zone in the departments of Beni and Santa Cruz in Bolivia, the use of combined land and river transportation is common. An investment of about 10 percent of the construction costs of a new road can permit farmers to use river transportation instead of new roads, resulting in a significant reduction in costs. This small investment is needed for constructing docks, parking lots, facilities for loading and unloading, and the like. On the other hand, passenger transportation costs for river transportation are significantly higher (4). These references indicate that if the ADT is equal to, or less than, 55 vehicles per day and if it mainly includes passenger pickups, it is not usually feasible to construct a new road. It will be more economical to use the river as a link in the rural transportation network until the traffic volume increases.

When the entire rural transportation network of roads and river navigation is evaluated, the flows of people and cargo are assigned to each link so that all the productive area is covered adequately and economically. Each road or river link serves the optimum area of influence, and the entire network covers all the productive areas. Generally, when there are no topographical or environmental obstacles, the area of influence of each road segment extends approximately 2 to 3 km. The extension of the area of influence is determined to permit local farmers to bring their products from the farthest point of their farms to the road during the day the products are harvested, preferably in less than 4 hr. Special local conditions such as difficult mountainous terrain, rivers, wetlands, natural resources, or protected flora/fauna may affect the actual area of influence on road links and therefore the planning and administration procedures of the rural transportation network.

Value Added from an Increase in the Area Under Cultivation

This value added is generally small and varies between 0 and 15 percent (4). As an example, the area influenced by the road distance extends 2.5 km in both directions (i.e., 500 ha/km of the road link). Assume that the average marginal increase in the cultivated area is (7.0 percent) (500 ha) = 35 ha/km. The value added for cacao (4) is U.S. $630 and U.S. $1,750/km/year for traditional and semitechnical farming methods, respectively.

Other Value Added

Benefits result from eliminating losses in existing crops caused by lack of access, poor surface conditions, and lowering of storage requirements and related inventory costs. This value added includes the benefits of eliminating the lack of accessibility to markets, having better agricultural products, and harvesting the crops when they are ready for marketing, regardless of weather conditions. More explanation of the way road accessibility is improved and leads to increased value added is given elsewhere (3).

ECONOMIC ANALYSIS

The purpose of the economic evaluation is to ensure that projects are only selected when the preceding factors generate more benefits than the total expenditure required during the lifetime of the project. When the internal rate of return exceeds the opportunity cost of capital, the road investment is normally considered economically justified. Each cost and benefit item is determined uniquely to ensure that there is no double counting. As an example, the reduction of the VOC is credited only to the transportation value added and not to the improvement of the agricultural farming system or to the reduction of production costs. The stream of economic costs and benefits is calculated for each transportation link and for the entire network, determining the related economic indicators such as IRR, FYRR, and NPV. The FYRR of each uniform link segment is used to determine the optimum year of rehabilitation and to ensure that the improvement of high-return roads is not delayed to accommodate barely feasible roads. The final order of priority for road improvement should also include social consideration and an evaluation of the capability of the local government and the local construction industry to carry out the projects as needed.

SOCIAL CONSIDERATIONS

South American governments and international finance agencies, such as the World Bank and IDB, specify that the results of the economic evaluation must be analyzed together with social factors (1,2,9). Population and the rate of illiteracy in the influence area of the local network are among the other social indicators used to determine investments of road improvements. The higher the population density and the higher the rate of illiteracy, the greater the need for transportation to local markets, public institutions, health and educational facilities, and commercial centers. For any given investment, the social benefits achieved from rural road improvement will be greater for a higher population density and for higher illiteracy rates (3,4,10). The distribution of net economic benefits accruing to low-income groups is another important social indicator. The definition of low-income groups and the procedures to measure this social indicator are presented in guidelines for the preparation of loan applications of IDB (1).
IDB recommends the investigation of several other concerns related to the impact of transportation on the human environment, such as safety aspects and impact on land prices, historical or archeological sites, and indigenous groups (1,2,17,12). These guidelines emphasize the following social issues:

- Upgrading of unpaved rural roads to paved standards should consider the demand and volume of slower-moving traffic that may be either displaced or put at risk as trucks and cars travel at higher speeds. If necessary and feasible, slow-speed lanes or pathways should be provided.
- The increase in land values along improved roads may be accompanied by land speculation at the expense of local interests and cultural values.
- Penetration roads that bring Amerindian groups into contact with larger society can have serious sociocultural impacts, especially where traditional tenure and resource use patterns are disrupted, jeopardizing indigenous livelihoods and welfare.
- Effective management of development and resource use in areas to be served by transportation infrastructure is essential for successful implementation of plans that address environmental and related social issues.

ENVIRONMENTAL ISSUES RELATED TO IMPROVEMENT AND MAINTENANCE OF RURAL ROADS

Environmental issues play an important role in the planning and administration of low-volume roads (2). On each project, it is necessary to identify the potential environmental impacts at the beginning of the project cycle and to classify the proposed activity in one of four categories according to the potential impacts, as follows: (a) beneficial to the environment, (b) neutral to the environment, (c) moderate potentially negative environmental impacts (but sound solutions exist for protection or remediation), or (d) significant potentially negative impacts. Road maintenance projects that improve surface and drainage conditions or that result in improvement of dust control may be classified as beneficial or neutral. Road improvements that require wetland replacement or erosion control may be classified in the third or fourth categories. The conclusions of the environmental analysis are implemented in the special provision of the construction documents to minimize or eliminate any damage or risk to people or the environment. A sample of potential negative impacts and the measures to mitigate them is presented in Table 2. As an example, to minimize erosion from fresh road cuts and fills, it is recommended to limit earth moving to the dry periods; protect susceptible soil surfaces with special mulch; protect drainage channels with berm, straw, or fabric barriers and install sedimentation basins; and seed or plant the susceptible erodible surfaces as soon as possible.

LOW-COST BRIDGES AND WATER CROSSINGS

It is well known that the transport of goods in some South American countries is mainly performed by the trucking industry, which saves money by overloading its trucks. Recent projects in Ecuador, Colombia, and Venezuela financed by IDB and the World Bank indicate that little is currently being done on the main roads to control truck overloading. As a result of this evidence, the structural divisions of some road authorities are using a 25 percent increase of live load, as compared with the AASHTO HS-20 standard of 33 tons. A lower standard live load of the 24.5-ton HS-15 truck was used only occasionally in Ecuador in the design of rural bridges. Recent economic and transport studies in Ecuador indicate that the actual vehicle loading on many rural roads is significantly lower (13). The largest vehicle on more than 90 percent of the roads is a two-axle truck with a total weight of less than 10 metric tons. About 75 percent of the vehicles are pickup trucks and light buses and trucks with a total weight of 2 to 6 tons. In addition, the projected demand and economic growth and the low standard of the road and pavement make the use of oversized or overloaded vehicles infeasible (3,4,10,13). Only in a few rural locations—regions with heavy traffic from timber-producing areas or banana plantations—can an AASHTO standard HS-15 live load be economically justified. These relatively few roads usually have higher design standards; a 6.0- to 7.2-m-wide base course pavement with or without blacktop is usually used. On the basis of these economic and traffic forecast analyses, it appears that it was practical and economical to adopt lower design standards for live loads on many of the low-cost rural bridges. The following three load categories were adopted (13): an M6 truck with 1200 and 4800 kg on the front and rear axles, respectively; an M10 truck with 2000 and 8000 kg on the front and rear axles, respectively; and an HS-15 load with 2720 kg on the front axle and 10 880 kg on each of the two rear axles, for a total of 24 480 kg. The AASHTO standard HS-20 live load was not found to be economically justified for these low-cost roads. Nevertheless, it is still used by the Colombian road authority for the design of rural bridges.

Hydrology design criteria may also play an important role in reducing construction expenditure on low-cost bridges. The recommended storm period in the design of bridges on rural roads is 25 years (13). The recommended clearance between the maximum storm water level and the bridge should be 1 ft unless the water velocity is very low. If the stream's ground slope is less than 0.5 percent, the maximum velocity is less than 10 ft/sec, and no accumulation of debris is expected, the water clearance could be reduced from 1.0 to 0.5 ft. Experience with design of low-cost bridges (13) shows that the average total cost of a one-lane low-cost bridge is approximately 20 to 40 percent of that of a standard two-lane bridge. Additional cost savings can be obtained by constructing a ford. Graveled fords are commonly used in the mountainous and hilly regions (13). Fords are used as low-cost water crossings on almost every unpaved rural road in this region. The construction of this type of crossing is usually labor-intensive. The surface of graveled fords usually performs adequately for 3 to 5 years in the Ecuadorian Andes. Maintenance is simple and is performed by manual labor with a relatively minor cost. Experience in Ecuador clearly indicates that the construction and maintenance costs of fords are always a small fraction of those for single-lane, low-cost bridges. They cost U.S. $50 to U.S. $100 per linear meter, or less than 5 percent of a two-lane standard bridge when local materials are available.
<table>
<thead>
<tr>
<th>POTENTIAL NEGATIVE IMPACTS</th>
<th>MITIGATING MEASURES</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Direct: During Construction</strong></td>
<td></td>
</tr>
<tr>
<td>1. Erosion from fresh road cuts and fills and temporary sedimentation of natural drainage ways.</td>
<td>Limitation of earth moving to dry periods. Protection of most susceptible soil surfaces with mulch.</td>
</tr>
<tr>
<td>2. Air pollution from: construction site dust, rock crushing plants, asphalt plants.</td>
<td>Appropriate controls, such as considering wind intensity and direction in construction schedule.</td>
</tr>
<tr>
<td>4. Creation of stagnant water bodies in borrow pits, quarries, etc. suited to mosquito breeding and other disease vectors.</td>
<td>Assessment of vector ecology in work areas and employment of measures to avoid creating habitats.</td>
</tr>
<tr>
<td>5. Environmental and social disruption by construction camps.</td>
<td>Careful siting, construction and management of construction camps.</td>
</tr>
<tr>
<td><strong>Direct: Permanent</strong></td>
<td></td>
</tr>
<tr>
<td>6. Destruction of buildings, vegetation and soil in the right-of-way, borrow pit sites, waste dumps, and equipment yards.</td>
<td>Alternative alignments. Harvest and utilization of public domain forest resources prior to construction.</td>
</tr>
<tr>
<td>7. Interruption of subsoil and overland drainage patterns (in areas of cuts and fills).</td>
<td>Installation of adequate drainage works.</td>
</tr>
<tr>
<td>8. Landslides, slumps, slips and other mass movements in road cuts.</td>
<td>Route alignment to avoid inherently unstable areas. Design of drainage works to minimize changes in surface flows and adequate to local conditions, according to prior surveys. Stabilization of road cuts with structures (concrete walls, dry wall masonry, gabions, etc.).</td>
</tr>
<tr>
<td>9. Increased suspended sediment in streams affected by road cut erosion, decline in water quality and increased sedimentation downstream.</td>
<td>Establishment of vegetative cover on erodible surfaces as soon as possible. Establishment of retention ponds to reduce sediment load before water enters stream.</td>
</tr>
<tr>
<td>10. Marred landscape (scars from road cuts, induced landslides and slumps, etc.)</td>
<td>Tourist site access roads planned with regard for visual aesthetics. Grade limitations to avoid cutting and filling where scenery would be spoiled. Maintenance and/or restoration of roadside vegetation.</td>
</tr>
<tr>
<td>11. Contamination of ground and surface waters by herbicides for vegetation control and chemicals (e.g., calcium chloride) for dust control.</td>
<td>Reduction of use. Alternative (non-chemical) methods of control.</td>
</tr>
<tr>
<td>12. Accident risks associated with vehicular traffic and transport. that may result in spills of toxic materials.</td>
<td>Regulation of transport of toxic materials to minimize danger. Prohibition of toxic waste transport through ecologically sensitive area.</td>
</tr>
<tr>
<td>14. Unplanned or illegal cutting or land-clearing; long-term or semi-permanent destruction of soils in cleared areas not suited for agriculture; destruction or damage of terrestrial wildlife, etc.</td>
<td>Prior surveys and effective management.</td>
</tr>
<tr>
<td>15. Planned development and illegal invasion of homelands of indigenous peoples by squatters and poachers causing serious social and economic disruption.</td>
<td>Prior surveys and effective management.</td>
</tr>
</tbody>
</table>
OPTIMIZATION OF ROUTINE AND PERIODIC MAINTENANCE EXPENDITURES

The principal objective of rural road network administration is to provide adequate all-weather accessibility while minimizing user and maintenance costs. To achieve this goal, a road inventory is prepared to predict surface distress and to determine the schedule of the most effective routine and periodic maintenance activities. These activities are defined as specific work operations needed to remedy or reduce road deterioration and thus provide adequate accessibility. The organization of the work into discrete activities simplifies administration and minimizes maintenance expenditures. The prioritization and scheduling of the work activities should be implemented in a manner that minimizes the rate of surface deterioration, keeps surface roughness low, ensures an adequate pavement condition index, lowers traffic hazards, and minimizes overall maintenance expenditure and road user costs (14). Typical routine maintenance work includes such activities as pothole patching; cleaning and small repair work of drainage facilities such as bridges, culverts, and low-cost water crossings; vegetation control; and simple pavement repair such as cracking/rutting/edge failure. The intensity and frequency of routine maintenance work is determined from the type and severity of the road distress characteristics and the traffic projections. Periodic maintenance such as pavement strengthening is needed when routine maintenance is not effective in preventing uncontrolled pavement deterioration and to en-

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**SUBTROPICAL ZONE**

![Graph showing optimum pavement rehabilitation period (Tr) for Subtropical Zone.]

**TROPICAL ZONE**

![Graph showing optimum pavement rehabilitation period (Tr) for Tropical Zone.]

**FIGURE 1** Optimum pavement rehabilitation period (Tr).
ensure safe travel and low user costs. The most economical schedule of pavement strengthening is determined from quality or effectiveness of the routine maintenance work, traffic projections, and environmental and topography characteristics. The quality or effectiveness of the routine maintenance plays an important role in road conservation (14). The environmental classification refers to tropical, subtropical, and arid climate conditions, all of which are present in South American countries. The annual precipitation in the tropical, subtropical, and arid zones is generally 2500 to 5000, 600 to 2500, and less than 600 mm/yr, respectively.

The optimum timing of periodic pavement strengthening is the one that results in minimum road user and conservation costs during the economic lifetime of the road. Figure 1 shows the relationship between the optimal periodic pavement strengthening, traffic volume in terms of ADT, routine maintenance effectiveness level (RML), and topographical and the environmental characteristics of gravel roads (14). For example, when the road has ADT = 80 and is adequately maintained (RML = 1.0) in a subtropical area, pavement overlay will be needed after 5 and 6 years for mountainous and level terrains, respectively. If the road has been very poorly maintained (RML = 0.0), pavement overlay is needed every 2 years. When only approximately 50 percent of routine maintenance work activities are properly carried out, pavement overlay should be carried out every 4 years. In subtropical areas, when ADT = 80 and RML = 0, pavement rehabilitation should be carried out once a year and more often in tropical areas.

CONCLUSIONS

1. To improve the performance of existing rural road networks and to optimize investment expenditures, a socioeconomic methodology can be used to determine the most economical investment program on the basis of the costs and benefits related to levels of accessibility, traffic volumes, and the economic life of the road. The paper presents eight road works and to optimize investment expenditures, a socio-economic methodology can be used to determine the most economic or effectiveness of the routine maintenance work, traffic projections, and environmental and topography characteristics. The quality or effectiveness of the routine maintenance plays an important role in road conservation (14). The environmental classification refers to tropical, subtropical, and arid climate conditions, all of which are present in South American countries. The annual precipitation in the tropical, subtropical, and arid zones is generally 2500 to 5000, 600 to 2500, and less than 600 mm/yr, respectively.

2. Good road management requires continual updating of information about the road network. This is achieved by means of a road inventory that identifies each road link and evaluates both engineering characteristics and the condition of the road elements. The inventory team has to make accurate assessments of at least 150 road engineering indicators. Logic analysis expert systems can be used to assist with this process and, in particular, ensure greater uniformity and reliability in the results.

3. On low-volume road networks, transportation cost savings may not justify rural road improvement, and the methodology should include the relationship between road accessibility, agricultural production, and economic and social indicators for the rural improvement. Rural investment can only be optimized when the most economical type of road network and the complementary agricultural investments are determined simultaneously.

4. Environmental issues play an important role in the administration of low-volume roads. For each project it is necessary to identify the potential environmental impacts and to classify the proposed activity in one of four categories: beneficial to the environment, neutral to the environment, moderate potentially negative environmental impacts (but sound solutions exist for protection or mitigation), or significant potentially negative impacts. Road maintenance projects that improve surface and drainage conditions or that result in improvement of dust control may be classified as beneficial or neutral. Road improvements that require wetland replacement or erosion control may be classified in the third or fourth categories. The conclusions of the environmental analysis are implemented in the special provision of the construction documents to minimize or eliminate any damage or risk to people or the environment. A sample of potential negative impacts and the measures to mitigate them are presented in the paper.

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REFERENCES


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Liquid Level Gauge for Measuring the Cross-Sectional Deformation of Aggregate-Surfaced Roadways

David A. Sacco, Ronald L. Copstead, and Donald J. Janssen

As part of an operational test assessing the effect of tire pressure on aggregate roadway deterioration, an instrument was developed to monitor deformations of the surface and subgrade at selected cross sections. This liquid level gauge measured elevation differences with a pressure transducer and recorded field data for processing on a personal computer. Heavy-duty hoses placed on the subgrade before the test allowed the passage of the transducer housing and permitted measurement of subgrade elevations at regular intervals across the roadway. Data from this device were plotted to show cross sections of the roadway structure with a precision of 3 mm (0.1 in.) for subgrade readings and 8 mm (0.3 in.) for surface readings. Surface and subgrade deformations were evaluated at weekly intervals and before and after maintenance activities. Rut development was traced, and contributions of subgrade and surface deformations were isolated. The instrument was portable, compact, and not affected by field or environmental conditions.

Recent developments in the availability of central tire inflation systems for use on commercial trucks in the forest industries have spurred the study of road deterioration as a function of tire pressure. The USDA Forest Service has been a leader determining the effects of tire pressure on roadway surfaces. As part of this effort, a liquid level gauge was designed, constructed, and used to monitor the deformation (surface and subgrade) of several aggregate-surfaced roadways at two test sites in Washington and one in Oregon. These tests were part of a larger effort to assess the effects of tire pressure on aggregate-surfaced roadways during operational log hauling and were a continuation of testing done by others to collect this type of data.

Roadway surface profiles and cross sections have been plotted using a number of methods, usually involving some type of optical or photogrammetric survey. Vertical subgrade deformations have been monitored with vertical probe extensometers, subsurface settlement points, or differential pressure gauges. The liquid level gauge described here is of the latter type and presented several advantages that led to its selection, development, and use. Subgrade and surface measurements were electronically recorded in a consistent format transferable to personal computers and compatible with spreadsheet software. Data were collected by one person without the need of an assistant, who would have been required with rod-and-level surveys. Unlike equipment that would have been permanently installed at specific locations within the pavement structure, the level gauge allowed deformations to be monitored at any desired number of points across the surface and subgrade at the selected cross section. The method described also proved to be fast, repeatable, and insensitive to site conditions such as weather.

This paper describes the development, design, and performance of the liquid level gauge and how it was used to measure subgrade and surface cross sections on aggregate-surfaced forest roads. Examples of cross-sectional deformation under heavy truck traffic are examined to illustrate the usefulness of the liquid level gauge in evaluating roadway performance.

BACKGROUND

Surface rutting is commonly used as an indicator of the performance of aggregate-surfaced roadways and, in the case of the operational test sites here discussed, as a trigger for maintenance activities. Many procedures for the design of aggregate-surfaced roadways (4–6) return the thickness of surfacing material required to prevent ruts of a given depth from forming under a certain quantity of traffic. Subgrade and surface strength are often entered as California bearing ratio (CBR) values or resilient moduli, and estimated traffic over the desired life span is entered in units reflecting both the magnitude and quantity of loading (such as 18-kip equivalent single-axle loads). A design chart, program, or nomograph establishes the thickness of surface rock required to prevent a rut of greater than 25 to 50 mm (1 to 2 in.) (typically) from forming under the specified traffic. Often these models assume that the subgrade, generally the weaker layer of the two, is vertically deformed under the wheel tracks due to stresses from traffic loading. This rutting is slowed or delayed by the dispersion of bearing stresses by the stiff surface material over the weak, underlying subgrade. Surface rutting is assumed to occur as the applied surface rock settles into the depressions developing in the subgrade.

The need to evaluate subgrade deformation arose in response to this notion of subgrade rutting as the underlying cause of roadway deterioration. Another concept of roadway performance was developed that anticipated near-surface rutting under high-pressure traffic due to shearing in the applied rock and surface and subgrade rutting under low-pressure traffic due to compaction or failure under traffic. The liquid level gauge, or some functional equivalent, was required to
distinguish the components of surface rutting attributable to deformations occurring in the surface material and subgrade.

**DESCRIPTION**

The liquid level gauge consisted of a pressure transducer connected by liquid- and air-filled tubes and control circuitry to an open liquid reservoir and a data recorder. It was used to establish the elevation of points along roadway cross sections relative to a fixed reference. Surface measurements were made by placing the transducer housing directly on the road surface. Measurements on the subgrade were made by passing the transducer and its umbilical through an empty, heavy-duty hose placed on the subgrade during roadway construction, before the application of traffic (Figure 1).

The instrument determined the relative elevation of a point on or beneath a road surface by measuring the pressure caused by a column of liquid. The height of the column was bounded by the free surface of the liquid in a reservoir and by a pressure transducer at the point of interest. The pressure was measured by a pressure transducer connected by a liquid-filled tube to the open reservoir. A second, empty tube provided an atmospheric pressure reference. By establishing the pressure difference between these two, the transducer determined the weight of liquid vertically between itself and the free surface at the reservoir. This weight was translated into a length on the basis of the specific gravity of the liquid and indicated the distance of the transducer (and the point of interest) above or below the free surface at the reservoir (Equation 1, Figure 1).

The level gauge used a solid state, piezoresistive, temperature-compensated, instrumentation-grade pressure transducer, 11 mm (0.45 in.) in diameter and approximately 41 mm (1.63 in.) long. It was sealed in a piece of thin-walled brass tubing 76 mm (3.0 in.) long with 13 mm outside diameter using epoxy. Two lengths of clear vinyl tubing, 6 mm (0.25 in.) outside diameter, terminated at the pressure transducer inside the brass housing. One length of tubing was filled with a solution of ethylene glycol in water (25 percent by weight) and was connected directly to a port on the pressure transducer. The other length of tubing was left empty and provided atmospheric pressure to the transducer. A precision thermistor was also enclosed in the brass probe housing. This allowed temperature to be recorded simultaneously with pressure, so the effect of temperature on the pressure transducer could be factored out. Electrical leads from the pressure transducer and thermistor were brought back along the tubes and connected to a readout and data logging unit. The entire apparatus was battery powered and designed to be portable (Figure 2).

The transducer was capable of reading pressure differences up to 13.8 kPa (2 psi). The liquid used had a specific gravity of 1.03 and a viscosity ranging from 0.0022 N·s/m² (0.46 × 10⁻⁴ lbf·s/ft²) at 20°C to 0.0035 N·s/m² (0.73 × 10⁻⁴ lbf·s/ft²) at 0°C. This liquid-transducer combination gave an elevation difference range of 1.66 m (65 in.). Calibration measurements were done each time the instrument was used so that uncompensated variations due to temperature or evaporation could be eliminated.

Mineral oil was initially used in the gauge, but the viscosity of the mineral oil (0.0331 N·s/m², or 6.91 × 10⁻⁴ lbf·s/ft², at 25°C) and the elasticity of the tubing seemed to cause large instabilities in the displayed readings. Readings stable to within 1.3 mm (0.05 in.) were achieved in 3 to 5 sec with the same tubing and the 25 percent solution of ethylene glycol, so the mineral oil was abandoned. Both the ethylene glycol solution and the mineral oil could withstand subfreezing to subtropical temperatures.

Before use, the tubes and electrical leads connecting the probe housing to the electronics box were taped together at 150-mm (6-in.) intervals. This made the "umbilical" easier to handle and indicated the spacing at which readings would be made in the subgrade. These tape bands were numbered beginning with zero at the probe housing.

The readout box contained power supply, signal conditioning, control, digitizing, data logging, and display components. The power supply and signal conditioning functions were provided on a single custom interface board. The control, digitizing, data logging, and display functions were performed by an off-the-shelf, battery-powered data logger. The interface board and the data logger were mounted in a single case. Field data could be uploaded directly to a desktop computer.

The data logger was configured to record several pieces of information at each reading: the date and time, a hose identification number, a position number, a temperature reading, and three elevation readings taken at 1-sec intervals. Multiple readings of the elevation were made to monitor and adjust for instabilities.

The probe housing and umbilical were designed to be threaded through a hose buried at the surface-subgrade interface to track deformations at that level of the roadway structure. Hydraulic pressure hoses [38 mm (1.5 in.) outside diameter; 25 mm (1 in.) inside diameter] with single- and double-braid steel reinforcement and wire-reinforced hydraulic suction lines were tested for resiliency by rolling over them with a pickup truck and a steel-drum roller. On the basis of observations, the suction hoses were eliminated.

![FIGURE 1 Liquid level gauge.](image-url)
FIELD INSTALLATION

Sections of hose 6.4 to 7.3 m (21 to 24 ft) long were placed across the roadway at points of interest. At one test site, they were laid on top of the prepared subgrade, staked down, and covered when the road was ballasted. At a second site, since the surface rock had already been placed, trenches were dug across the road down to the subgrade, hoses were placed, and the trenches were backfilled and compacted. Lengths of twine were threaded through the hoses and left there to pull the probe and its attached tubes and wires into position to make a set of readings. Subgrade hoses were kept capped during installation and between readings.

Steel fence posts were driven into the ground in line with and about 0.5 m beyond the hose ends to serve as supports for the open fluid reservoir of the liquid level gauge and as benchmarks. Rod-and-level surveys were done to tie together these reference stakes and to monitor overall slope stability. Notches were filed into the fence posts at about the original crown height of the road. These marks were used as reference locations for the free fluid surface in the reservoir and allowed the gauge’s range to encompass both subgrade rutting and surface heaving.

PROCEDURE

The fluid reservoir of the gauge was strapped to a post adjacent to a hose end so that the fluid surface was aligned with the reference notch in the post. The probe was attached to the preplaced twine in the hydraulic hose and drawn through until it was just protruding from the opposite end. The cord was left attached so that it would be pulled back through the hose, ready to use on the next visit. Returning to the reservoir (zero) end of the hydraulic hose, an L-shaped steel bar of known length [321 mm (12.63 in.)] was clamped onto the fence post so that its upper end was just touching the base of the fluid reservoir (Figure 2).
Calibration

A set of calibration readings was made immediately before and after each set of hose readings. After entering a hose identification number (ID) and a position number to indicate that the measurements were for calibration purposes, four readings were made with the probe stationary at the opposite end of the subgrade hose and the reservoir resting alternately on the top end and bottom end of the steel bar clamped onto the fence post. In effect, the length of the bar was being measured by the gauge, so that a calibration factor could be calculated by comparing the probe-perceived and actual lengths of the bar. Experience indicated that greater stability in readings was achieved by moving the reservoir and not the probe, since the reservoir was easier to fix securely to the stake and the probe housing would be stable in the subgrade hose. Following these readings, the hose number was entered as the ID, and the subgrade readings were begun.

Subgrade

The first reading made in the subgrade was assigned a position number corresponding to the number on the umbilical at the reservoir (operator) end of the subgrade hose. The data logger was preprogrammed to assign position numbers to subsequent readings in descending order until reset. This allowed the operator to compare the hose number with the position number in the data logger and avoid missed readings. The probe was drawn through the hydraulic hose 150 mm (6 in.) at a time, with a pause at each tape band for a reading. This generally required 12 to 15 sec, allowing for the reading to stabilize and for the data to be acquired. The final reading in the subgrade was made with the probe housing at the end of the hydraulic hose nearest the reservoir and operator.

Surface

Before making surface readings, a tape measure was stretched level across the roadway between the fence posts. The first reading was made in the far end of the subgrade hose opposite the reservoir. Readings were then made at 150-mm (6-in.) horizontal intervals along the roadway with the probe housing directly on the road surface. A final reading was made with the probe at the zero end of the subgrade hose. The probe was then threaded back 0.76 m (30 in.) into the hydraulic hose, and a second set of calibration readings was made. Since some of the hose ends protruded from the ground a short distance, the readings were made back far enough in the hose to ensure that the hose was secure, but not far enough into the hose that traffic effects would be significant. These readings also served as references in case any of the fence posts were disturbed.

Data Analysis

After reading all hoses at a site, data were downloaded to a desktop personal computer and translated into a text format readable by a spreadsheet program. Data reduction involved several steps and resulted in an x-y plot of surface and subgrade profiles for a particular hose. Data were first screened for extraneous readings and errors in assigning identification or location numbers. The three elevation readings were reviewed and reduced to one: any reading that occurred twice or more was accepted, or the average of all three was accepted if no value repeated. A calibration factor for that day’s readings was calculated and applied to all that day’s data.

The calibration factor was found by comparing the probe-measured bar length with the actual bar length. Each pair of top-of-bar and bottom-of-bar readings represented one instrument-measured bar length, and the calibration factor for the day was the actual bar length divided by the average of all of the gauge-perceived length measurements made on that day. Multiplying elevation readings by this factor corrected gauge-perceived length to actual length. No correction of elevation readings was made if the average calibration factor for that day fell between 0.99 and 1.01.

The level gauge did not measure horizontal positions. Since surface readings were taken at 150-mm (6-in.) horizontal intervals, the x-coordinate for any position was found by assigning a value of zero to the first reading. Subgrade readings were made at 150-mm (6-in.) intervals along the actual interface, and horizontal distances were calculated as shown in Figure 3. The x-coordinate for any subgrade reading was calculated as the sum of the horizontal distances between readings to that point, assuming a zero value for x for the first reading.

Subgrade elevations were interpolated at horizontal locations corresponding to the locations of surface readings. This eased the calculation of surface thickness and cross-sectional area and provided consistent points whose elevations could be tracked over time at both the surface and subgrade. A sample data plot is shown in Figure 4.

PERFORMANCE

Repeatability

To evaluate the repeatability of readings of the gauge, four sets of subgrade and surface readings were made at Hose 35 at the Vail, Washington, test site on May 22, 1992. These were preceded and followed by rod-and-level measurements of the surface at that location. The results of this field trial are given in Table 1. The precision of the level gauge was estimated as 8 mm (0.3 in.) for surface readings and 3 mm (0.1 in.) for subgrade readings on the basis of the difference
Readings at the subgrade were quite variable near the ends of the hose. At this location, the hose ends protruded some distance beyond the ground surface and were able to flex and deform significantly. Readings adjacent to the hose ends ranged up to 70 mm (2.8 in.), whereas readings made 0.75 m (30 in.) or more in from the hose ends ranged over 5 mm (0.2 in.) or less. Because of this, and the variability in surface roughness across the road surface, readings within 0.75 m (30 in.) of the hose ends were excluded in estimating precision. The precision of the probe for subgrade measurements would have been improved if a satisfactory means had been found to keep the probe housing centered or resting securely within the subgrade hoses. Efforts to find satisfactory means were unsuccessful.

The repeatability of surface readings was highly dependent on the nature of the surface rock. The roads evaluated during this operational test were constructed with pit-run shot rock. In the trafficked portion of the roadway, rocks were broken down and the surface was well choked with smaller particles, whereas boulders, coarse gravels, and significant void spaces were present on the shoulders. As a result, elevation readings from the central portion of the roadway tended to be more replicable than data from the edges. Readings taken at any particular point in the wheel tracks generally ranged up to 5 mm (0.2 in.), and readings at any point on the shoulders varied up to 31 mm (1.2 in.).

Surface readings made with the level gauge for this trial compared favorably with those made by rod-and-level surveys immediately before and after the gauge readings. The range of surface readings attained with the level gauge during this test was slightly higher for all positions than that achieved by rod-and-level surveys and lower than the optical method when the shoulders were excluded. The roadway cross sections as defined by the two methods were quite similar.

Temperature Sensitivity

The liquid level gauge was used on three sites from November 1991 through June 1992 in temperatures ranging from 2°C to 40°C, as measured by the thermistor in the probe housing. Calibration readings made over this period with two gauges indicated that there is a slight increase in the elevation difference measured by the probe with temperature, in spite of the compensation made by the transducer. Analysis of absolute error (AE) versus temperature (T) indicated the following:

\[
AE = -0.2248T + 4.4137 \quad r^2 = 0.5146
\]

The low \( r^2 \) is mainly attributable to improper (out of plumb) placement of the steel gauge bar on the fence post or to imprecisely seating the reservoir on the gauge bar, since calibration factors tended to be consistent at any particular hose location. Over the range from 5°C to 20°C, where most readings were made, absolute error varied from roughly 3 mm (0.13 in.) down to virtually zero. This corresponded roughly to the estimated precision of the instrument.

The gauge was used in weather conditions ranging from cold, driving rain to hot sun. Provided that the electronics case was protected, the gauge’s performance was not impaired. Some dramatic fluctuations in readings were attributed to wind blowing over the open ends of the air and fluid tubes, altering the pressures perceived by the transducer. This

\[
\begin{array}{|c|c|c|c|c|}
\hline
\text{Reading Method} & \text{Location} & \text{Median Standard Deviation (mm)} & \text{Maximum Range (mm)} & \text{D2S Precision (mm)} \\
\hline
\text{Hose 35, Vail Test Site, 4 Repetitions at 43 Positions} & & & & \\
\text{Level Gage} & \text{Surface} & 3.2 & 31.0 & 9.1 \text{ (1)} \\
\text{Rod & Level} & \text{Surface} & 2.2 & 25.9 & 6.1 \\
\text{Level Gage} & \text{Subgrade} & 1.0 & 70.2 & 2.9 \\
\hline
\text{Hose 35, Vail Test Site, Outer 5 Positions Excluded, Each End (2)} & & & & \\
\text{Level Gage} & \text{Surface} & 2.8 & 22.9 & 7.9 \\
\text{Rod & Level} & \text{Surface} & 2.2 & 25.9 & 6.1 \\
\text{Level Gage} & \text{Subgrade} & 1.0 & 5.0 & 2.7 \\
\hline
\end{array}
\]

1. Precision estimated by methods described in ASTM Practice C 670, for Preparing Precision Statements for Test Methods for Construction Materials.

2. Outer 5 locations excluded due to variability associated with subgrade hose flexure or highly irregular surface. See discussion on repeatability for comments.

FIGURE 4 Sample data plot (Hose 12, June 7, 1992).
was remedied by protecting both tube ends from the wind without cutting them off from ambient pressure conditions.

**Consistency of Readings over Range**

Elevation readings were taken over almost the entire range of the pressure transducer during the monitoring program. Some sets of readings ranged over 0.76 m (30 in.) because of the addition of surface material during logging operations and the heaving of surface rock in the center of the roadway. To determine whether the probe measured consistently over its range of about 1.65 m (65 in.), readings were made with the probe housing stationary and the reservoir held at 77 mm (3 in.) increments along a steel rule mounted vertically. An analysis of absolute error (AE) versus elevation difference (ED) indicated the following:

\[ AE = -0.0019ED + 0.1917 \quad r^2 = 0.2090 \]

Absolute errors deviated from this best-fit line by up to 3 mm (0.12 in.). For a surface cross section with an overall elevation difference of 0.6 m (24 in.), this would have resulted in a potential error of 6 mm (0.24 in.), within the estimated precision of the instrument.

**Rotational Orientation of Pressure Transducer**

A question arose as to whether readings would be altered by the orientation of the transducer around its longitudinal axis within the hydraulic hose, particularly since this would be beyond the operator’s control. It was not known how precisely the transducer had been mounted within the housing or whether it might have been sensitive to having the air or fluid hose “up.” A test was run to evaluate this by taking a series of readings on a level surface and rolling the probe through 45 degrees after each. This test revealed an eccentricity in the probe housing of 1.3 mm (0.05 in.), less than half the estimated precision of the gauge.

**EXAMPLE ANALYSIS**

In the first phase of the operational test near Raymond, Washington, log trucks made 236 round-trips (empty and loaded passes) on high-pressure tires [600 to 700 kPa (90 to 100 psi)]. The road had 450 mm (18 in.) of surface material (USCS GW, PI 4, less than 1 percent fines, 100 mm top size) over a sandy subgrade (USCS SP, PI 11, 3 percent fines). The in-place CBR of the subgrade was estimated (8) as 24 by dynamic cone penetrometer (DCP) tests, which gave a DCP index of 10 to 12. The in-place CBR of the surface was estimated (9) as 30 by Clegg hammer readings, which gave a Clegg impact value (CIV) of 18 to 22. The weather was cool with frequent rains. Figure 5 shows several plots of surface and subgrade profiles made during the 5-week period of traffic. Ruts 70 to 150 mm (3 to 6 in.) deep developed in the wheel tracks along with surface heave along the centerline and lateral displacement of the shoulders. Minimal deflections occurred in the subgrade. This pattern of deformation suggests that a bearing capacity failure occurred in the surface material because of the high near-surface stresses.

Figure 6 shows several sets of data taken at critical intervals at Hose 32 on the upper test road at Vail, Washington. Subgrade CBR was estimated as 10 (DCP 4 to 5), and surface rock with a CBR of 90 (CIV 35 to 40) was applied 230 mm (9 in.) thick. Surface rock was USCS GW, PI 4, less than 1 percent fines, 100 mm top size. Subgrade material was USCS SP, PI 12, 2 percent fines. This cross section was exposed to 447 passes each of empty and loaded log-haul vehicles using tire inflation pressures of 350 to 400 kPa (50 to 60 psi). The weather over the 10-week period of the test was predominantly dry and cool with occasional rain. No significant maintenance was performed on this section of roadway during the test. Modest deformations can be seen developing in both the surface and subgrade as the test progressed.
CONCLUSION

The liquid level gauge developed in this work is a reliable tool for monitoring vertical roadway deformations with a precision of 3 mm (0.1 in.) for subgrade readings and 8 mm (0.3 in.) for surface readings. It appears not to be affected by a wide range of weather and temperature conditions, nor does its accuracy appear to change over its working range. The instrument proved very useful in establishing different trends in aggregate roadway performance under heavy vehicles using different tire inflation pressures.

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REFERENCES


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